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A STUDY OF THE SWELLING BEHAVIOR OF AN EXPANSIVE CLAY AS INFLUENCED BY THE CLAY MICROSTRUCTURE, SOIL SUCTION AND EXTERNAL LOADING

Phil V. Compton
Capt USAF

TECHNICAL REPORT NO. AFWL-TR-70-26

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AIR FORCE WEAPONS LABORATORY
Air Force Systems Command
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FOREWORD

This research was performed under Program Element 63723F, Project 683M, Task 49005.

Inclusive dates of research were May 1969 through January 1970. The report was submitted 3 March 1970 by the Air Force Weapons Laboratory Project Officer, Captain David D. Currin (WLCT-A).

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This technical report is the result of research performed at the Graduate College of Texas A&M University in fulfillment of the requirement for the degree of Doctor of Philosophy for Captain Phil V. Compton.

This technical report has been reviewed and is approved.

David D. Currin

DAVID D. CURRIN
Captain, USAF
Project Officer

Clarence E. Teske

CLARENCE E. TESKE
Major, USAF
Chief, Aerospace Facilities Branch

CLIFF M. WHITEHEAD
Colonel, USAF
Chief, Civil Engineering Division

ABSTRACT

(Distribution Limitation Statement No. 2)

Influence of soil suction, clay microstructure, and applied external loading upon the swelling behavior was undertaken to further the understanding of expansive soils. For the soil tested, the microstructure produced by kneading compaction did not change the particle orientation to any significant degree. Evaluation of the microstructure was performed with both X-ray diffraction and scanning electron microscopy techniques. The microstructure is perhaps best described by multi-grain or packet arrangements. A higher degree of packet dispersion occurred at molding water contents wet of optimum; however, it did not approach any high degree of preferred particle orientation. A swell test was developed using the Bishop Oedometer in which both soil suction and external loading could be controlled and measured. Using the axis of translation technique with a high air entry ceramic stone, soil suctions of 5.1 kg/cm^2 (pF≈3.4) were measured. Results indicate that below a certain value of soil suction the swell will be accelerated with a further decrease in suction. As the applied loading is increased it will limit this acceleration and the soil suction-swell relationship becomes more linear with decreasing suctions.

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SECTION I

INTRODUCTION

Problem Soils which are potentially expansive occur throughout the world, however, those located in the arid and semi-arid regions are the most active. The soils engineer, who must construct on or with these expansive soils, must be able not only to identify them, but also to predict their expansive behavior in order to successfully design a structure whether it be a road, airfield or building.

Currently, sufficient knowledge is available to properly identify those soils which are potentially troublesome. However, the accurate prediction of the degree of expansion that can be expected during the life of the overlying structure is not very successful. The annual property loss from damages caused by expansive soils has been conservatively estimated to be over one billion dollars (1).

Whether or not the soils will actually expand, and how much they will expand depends on many items which can be grouped basically into (a) environmental factors and (b) physical factors involving the soil itself.

The one basic requirement for expansion to occur is a change in moisture content. This change is dependent upon the unsatisfied "thirst" of the soil as influenced by its environment and certain

physical properties of the soil. This water demand of the soil is best described in terms of soil suction. It is known that the release of existing suction pressures in expansive soils (by the addition of water) and the magnitude of the applied external loads are important factors. Standard swell tests, now in use, are unable to control both of these variables and their respective influences cannot be assessed. In addition, it is logical that the arrangement of the soil particles, whether random or oriented, must also influence the magnitude of the soil swell.

Objectives To obtain a more complete picture of the factors influencing the amount of swell, it is necessary to test soils in such a manner whereby particle orientation, the external applied load and the soil suction pressure can be independently controlled. The latter two factors can be controlled in the newly developed Bishop Oedometer, whereas particle orientation can be controlled by the method of sample preparation. It must be pointed out that this research is exploratory in nature and it is hoped it will serve as guidance to other researchers.

The specific objectives of this study are:

(a) To determine the influence of soil microstructure on the swelling properties of an expansive clay soil.

(b) To determine the effect of controlled suction pressure and external loads on the swelling behavior of an expansive clay soil.

(c) To determine what relationship, if any, exists between swell, soil microstructure, soil suction, and external loading.

SECTION II

GENERAL CONCEPTS AND TERMINOLOGY

The purpose of this section is to provide the reader with the definition of specific terms or concepts which continually reoccur in the text. The definitions provided are those which appear to be the most acceptable to the related scientific fields.

Adsorbed water. The adsorbed water consists of those water molecules attracted by the electrical charge on the clay particle.

Diffuse double layer. The attraction to exchangeable ions to clay particles in a clay-water mixture is opposed by the tendency of these ions to diffuse and distribute themselves evenly in water. The result is a diffuse grouping of ions about a clay particle. This system is considered to be a diffuse electrical double layer, one layer being the negative charge on the surface of the clay particle and the other layer formed by the ion concentration near the particle surface. The entire double layer is an electrical field and the water in the field is attracted by an induced electrical force.

The thickness or extent of the double layer is dependent upon several factors including concentration and charge of the exchangeable ions present in the system, temperature and pH of the solution.

Repulsive forces between clay particles. The repulsive forces active between two adjacent clay particles are created by the interaction of their diffused double layers. The double layers have the same electrical charge (usually positive) and act to repel each other as would two magnets of like charge. The strength of the repulsive forces is controlled by the thickness of the double layer.

The thicker the double layer the greater the force of repulsion. The extent of the double layer may range from 50 to 300Å (1).

Attractive forces between clay particles The forces of attraction between clay particles are predominantly the London-van der Waals forces. These attractive forces are between individual atoms and are relatively small in magnitude. The strength is rapidly dissipated with distance between particles.

Another source of the attractive forces may be caused by the occurrence of positively charged ends on the clay particles themselves, due to fracture planes which break the mineral in such a manner as to leave a net positive charge (2).

The clay micelle The clay micelle is a convenient model by which to visualize the clay particle and the diffuse double layer concepts. The adsorbed water is considered to be those water molecules attracted to the clay particle and the diffuse double layer is represented by the water and ions present in the micelle. The size of the clay micelle is controlled by those factors which influence the diffuse double layer previously covered. Fig. 1 shows the clay micelle.

Osmotic pressure in clay water system For osmotic pressures to exist two basic conditions must be satisfied. Two solutions of different concentrations must exist which are separated by a semi-permeable membrane which will allow passage of the solvent (water) but not the solute (ions).

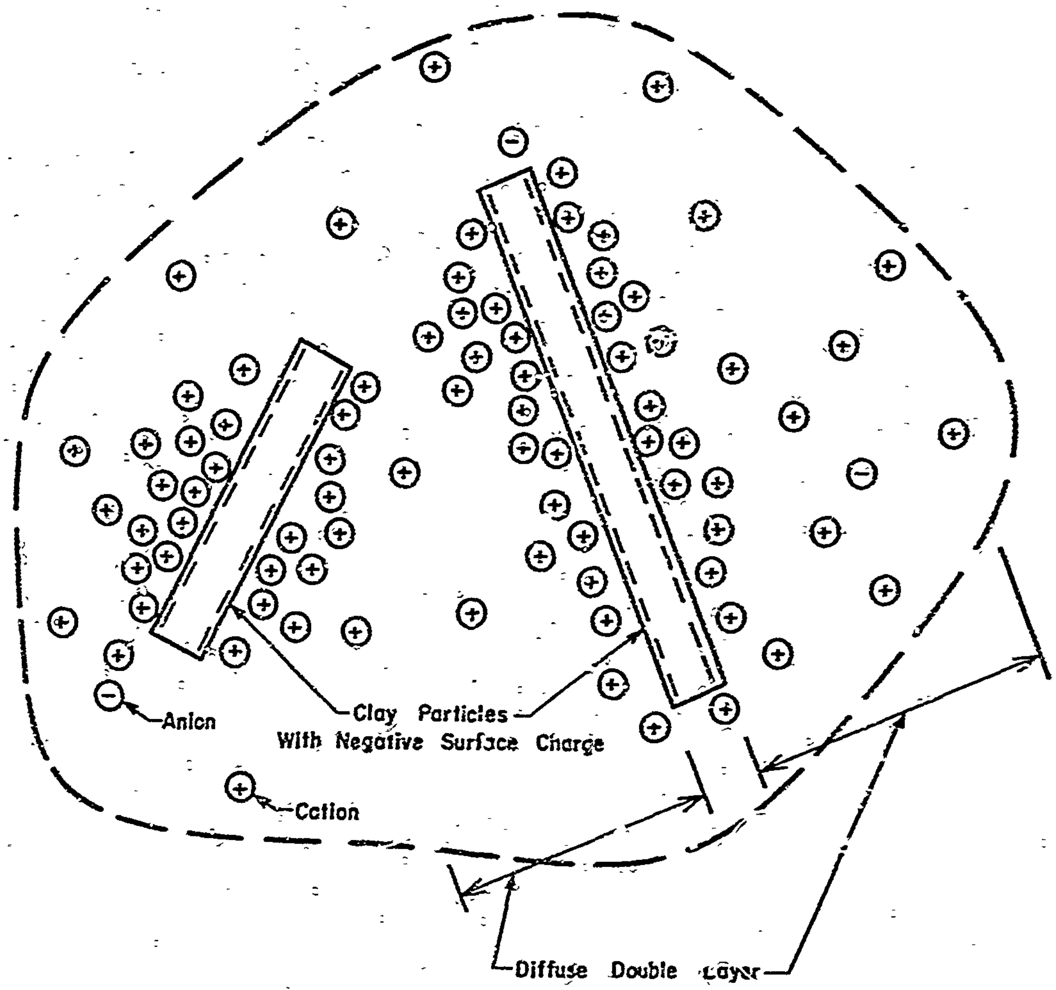


Figure 1. The Clay Micelle (After Woodward)

In examining the clay-water-ion complex in the double layer, the ions are held by strong electrostatic forces and are not free to move away from the double layer. However, water molecules outside the double layer (pore water) are free to move in and dilute the ion concentration. Thus, the double layer acts as a semi-permeable membrane and satisfies the requirements for osmotic pressures to exist in a clay-water system.

Osmotic suction The osmotic or solute suction has been defined as "The negative gage pressure to which a pool of pure water must be subjected in order to be in equilibrium through a semi-permeable membrane with a pool containing a solution identical in composition with the soil water." (3)

Capillary suction The matrix or capillary suction is defined as "The negative gage pressure relative to the external gas pressure on the soil water, to which a solution identical in composition with the soil water must be subjected in order to be in equilibrium through a porous permeable wall with the soil water." (3)

Soil suction The soil suction is the sum or capillary and osmotic suction pressure and is always a negative value with respect to atmospheric air pressure. The pF scale developed by Schofield (4) has been used to measure soil suctions. The pF scale is the logarithm to base ten of the suction expressed in centimeters of water. A suction of ten centimeters of water is equal to pF=1, one thousand centimeters is a pF=3, etc.

Clay microstructure The arrangement of the individual clay particles is referred to as the microstructure. The microstructure has been classified into two basic arrangements. A "flocculent" microstructure is characterized as containing a random orientation of particles with considerable "end to face" contact between particles. The other basic orientation is termed a "dispersed" microstructure and is characterized by a parallel orientation of the particles with a "face to face" arrangement.

SECTION III

REVIEW OF THE LITERATURE

For an expansive soil to swell the moisture content of the soil must increase. The degree to which the change in moisture content will change, given an opportunity by the environment, depends upon the soils demand for water.

The evaluation of the soils thirst for water is at best a very complex problem and a rigorous division of all the forces involved into individual components (adsorption, osmotic pressure, capillary menisci, air pressure, predominant cation, exchange capacity, etc.) has not been sufficiently developed to the point of practical application (1). However, the total net effect resulting from all of these forces has led to the use of soil suction as a method to evaluate a total demand of the soil for water.

Satisfying this thirst in an expansive soil will result in a volume change, the degree of which is dependent upon not only the reduction of the soil suction but also upon certain physical properties in the initial condition of the soil (density, microstructure, external loading). For simplification the literature pertinent to this study has been divided into two sections (a) soil suction and (b) physical properties of the soil.

<u>Soil Suction</u> In nature, the soil-moisture regime is in equilibrium with the environment even though this equilibrium may be a dynamic one. In a partly saturated soil, that is, a three phase

system of air, water, and soil, this equilibrium can best be described in terms of soil suction. The soil suction has been defined as the sum of capillary and osmotic suction pressures, and is always a negative value with respect to atmospheric air pressure.

The soil suction may reach very high values and is greatly influenced by small changes in moisture content. Values of soil suction up to 100 psi have been reported for soils in the range of moisture contents encountered in engineering work. In very dry soils values exceeding 100,000 psi may exist (5).

A suction profile is established in the partly saturated soil and is dependent upon several environmental factors which include depth to the water table, climate (rainfall and relative humidity), and vegetation. The climatic conditions are responsible for the majority of the changes in soil suction in natural soils. The arid and semi-arid areas of the world have the most pronounced seasonal variations in rainfall and consequently are the areas where expansive soils create the most problems (6). The depth of this zone of seasonal moisture fluctuations will vary from location to location and is dependent upon the type and condition of the soil, extent of seasonal variation, etc. The resultant effect of these moisture changes is a corresponding reduction (wetting) or increase (drying) in the soil suction, which may be accompanied by swelling or shrinking, respectively, in an expansive soil.

Once the soil surface is covered by an impermeable cover (road, building, etc.), this balance between climate and soil suction is changed. The loss of water by evaporation can no longer occur as it is prevented by the impermeable cover. A new suction profile will then occur over a period of time due to a gradual increase in moisture content under the vacility, resulting in a decrease in the soil suction.

The size and shape of the cover will also determine the extent to which the suction profile will change. In the case of long and narrow coverage (road or runway) the soil suction at the center remains less than the edges, while the edges are subjected to considerable fluctuations caused by seasonal climatic conditions. Under a large square building, on the other hand, the suction profile will remain more uniform during seasonal changes, however, the edges experience the same fringe fluctuations. A building is sensitive to man's influence by planting and watering of vegetation along the edges of a building, poor drainage of the area adjacent to the building or leakage of water from the piping system servicing the facility which will cause localized decreases in the suction profile. In all cases the development of different suction profiles in an expansive soil will lead to possible expansion or shrinkage, creating a differential volume change and resulting in damage to the facility.

While the change of soil suction in an expansive clay is known to have a direct bearing upon the resulting swell of the clay, the ability to define its influence has been severely hampered by the

lack of a convenient laboratory method to measure soil suctions exceeding about 12 psi, while at the same time subjecting the sample to an external load as it would be in the field (6). Consequently the role that changing soil suctions play in the swelling behavior has not been clearly defined. The importance of this factor was emphasized by Gibbs (7) in summarizing the "Status of the Art of Dealing with World Problems on Expansive Clay Soils" by stating

> . ."evaluation of the suction pressures of forces in unsaturated soil might be very closely related to the problems in expansive clays, and that one of the approaches to the problem of expansive clays would be to correlate these suction forces with heaving."

Measurement of soil suction The main difficulty in measuring suction pressures is that the water in the measuring system tends to cavitate at pressures of approximately -1 atmosphere. To overcome this limitation the technique of "Axis Translation" has been developed by Hilf (8). In this method, the pore air pressures are increased until the soil suction pressure can be measured. The radius of curvature of the air-water menisci does not appear to change significantly as the air pressure is increased, thus the difference between the final air and water pressures equals the initial suction pressure in the soil. Several investigators (8, 9, 10) have examined the accuracy of the axis translation technique and found it valid within the range of experimental error for the usual values of applied air pressures of less than 200 psi. Fig. 2 shows the experimental values obtained by Olson and Langfelder (5), the experimental line has a slope of 45.5° instead of 45° which is well within the range of experimental error.

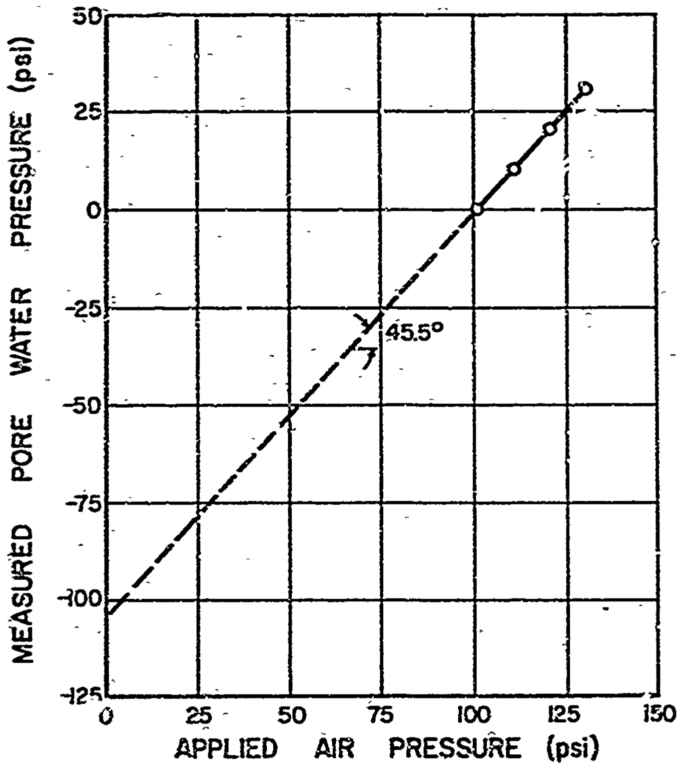


Figure 2. Measured Pore Water Pressure versus Applied Air Pressure (After Olson and Langfelder)

Gibbs and Coffey (11) used the axis translation technique in developing a method to measure initial suction pressures in specimens that were later used in triaxial shear testing. Their procedure used a pressure cell in which an all around air pressure could be applied to the soil sample. The test specimen did not completely cover the high air entry ceramic stone so that the air pressure also acted on the ceremic stone. They observed that about 75% contact between stone and soil gave reasonable time periods for equilibrium to be reached between the applied air pressure and measured water pressures.

Olson and Langfelder (5) studied the soil suction of compacted clays. They observed very high values of soil suction on the dry side of optimum moisture content. The slope of the curve in Fig. 3 is of interest as it illustrates how small differences in molding moisture content can greatly influence the resulting soil suction.

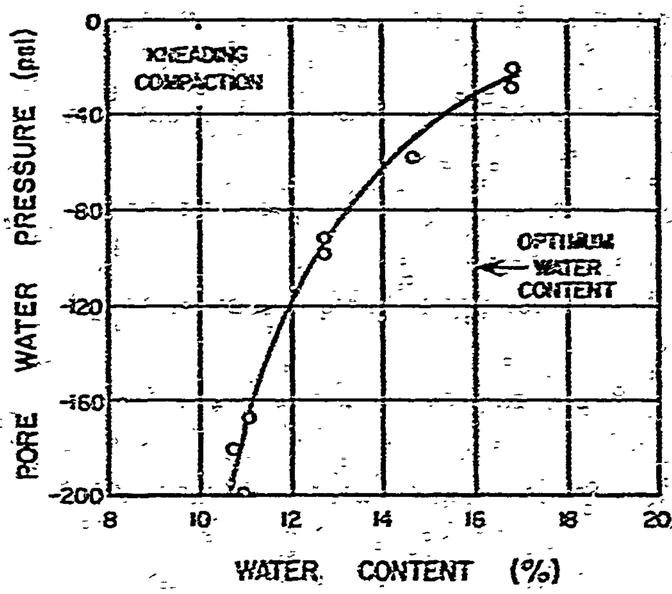


Figure 3. Measured Pore Water Pressure versus Water Content for Kneading Compaction (After Olson and Langfelder)

<u>Soil suction influence in swell</u> The correlation between soil suction and swell is not well documented in the existing literature. The main emphasis has been to relate the change in moisture content which, while important, may not be as significant as the

initial soil suction at that moisture content. Very small changes in moisture content (1 to 2%) can cause detrimental swelling under light foundations (12). These small changes in moisture content have a significant effect upon the soil suction, and perhaps the soil suction, or more specifically the change in soil suction, would provide a more exact parameter in determining the swelling potential of expansive soils.

In analyzing the swell as the soil suction is released there appears to be two main mechanisms that cause swell. First, the reduction of capillary tensions may contribute a more mechanical type of swelling, that is, the elastic rebound of clay particles which may have been deformed at high values of soil suction. The magnitude of this component of swell is normally small (13).

The second mechanism is that of the uptake of water molecules by the double layer to satisfy the osmotic pressures created by the imbalance in ion concentrations in the clay micelle and pore water. The addition of water to the clay micelle increases its size and therefore creates swell. This type of expansion is referred to as osmotic swell and is perhaps the most important mechanism causing swell (1). The soil micelle will continue to take on additional water molecules pushing the particles further apart until the osmotic pressures are satisfied or until they come into equilibrium with the forces tending to push the particles together.

In comparing the relative strengths of osmotic and capillary pressures, Mitchell (14), has shown that for relatively large pores

(0.5μ) the two forces are of the same order magnitude, while smaller pore sizes below 0.1μ, the capillary pressures may be on the order of ten times that of the osmotic pressure.

The influence of osmotic swell has been shown by several investigators by using compacted clays (15, 16). The swell for samples compacted wet of optimum moisture content has been reduced or completely prevented by allowing the sample to imbibe water with a high ion concentration, thus, demonstrating that when the double layer demand of the clay is nearly satisfied (as is the case of wet of optimum molding water contents) the potential swell is primarily a function of osmotic demand to balance the clay micelle water and pore water ion concentration.

The compacted clay samples dry of optimum have been shown to experience significant swell. The influence of retarding swell by high ion concentration in the pore water is less effective. However, considering that other mechanisms are involved this would appear logical. Ladd (15) in analyzing this condition believes that a combination of factors must contribute to the swell. The incomplete development of the double layer, the effect of the electrical force field on water molecules, effect of cation hydration as well as the elastic rebound of deformed particles could all contribute to the swell.

As pointed out earlier, the relationship that reducing soil suction has on swelling behavior has not been defined under loaded conditions. However, the recent development of the Bishop Oedometer, primarily as an improvement to the standard consolidation test, has

made it possible to perform swell tests under measured or controlled soil suctions while subjecting the specimen to controlled external loads. The range of soil suction pressures may be increased by using the axis translation technique. The oedometer is modified by replacing the normal porous stone with a high entry ceramic stone which has an air entry value greater than the applied air pressures to prevent failure of the measuring system.

<u>Summary</u>

1. The soil's total demand for water is best expressed in terms of soil suction. The individual components creating soil suction pressures have not been sufficiently developed to the point of practical application.
2. Any change to the existing equilibrium between the suction in a soil and its environment (impervious cover, loading) will change the suction profile. This change may result in a volumetric increase or decrease of the mass.
3. The influence that a reduction in suction has upon an expansive soil is known to have a direct bearing upon the swelling behavior. To date correlation of soil suction and swell has been limited due to lack of techniques to investigate this influence.
4. The axis translation technique of increasing the pore air pressure has been a successful method of measuring soil suctions. Limitations to the magnitude of the soil suctions measured are dependent upon the air entry pressure of the

ceramic stone used. Initial soil suction pressures up to 200 psi have been reported.

Physical Properties The physical condition will play an important role in determining the amount of volume change in expansive soils. The physico-chemical properties including the influence of the type and amount of exchangeable ions present in the soil and the type and amount of clay minerals, while very important, have been the subject of recent investigation and the reader is referred to Holt (17) for a comprehensive treatment. This section will concern itself more with those properties which effect swell considering the initial condition of the soil. The more important factors include soil density, external loading and the soil microstructure (13).

Density The soil density is an important factor which influences the amount of volume expansion. The higher the density (closer particle spacing) the more particles are contained per unit volume. This causes a higher degree of interaction between the clay particles; double layers and therefore higher repulsive potential between particles. As water is imbibed into the soil these repulsive forces push the particles apart and result in a larger expansion as the initial density becomes greater. The change in swell with respect to density for a compacted clay is shown by Fig. 4. Also illustrated is the influence of the molding water content. Comparing the densities produced at a moisture content of 20%, it can be seen that a density of about 95 lbs/cu ft has a volume change of 6% whereas a density of 80 lbs/cu ft swells only about 3.5%.

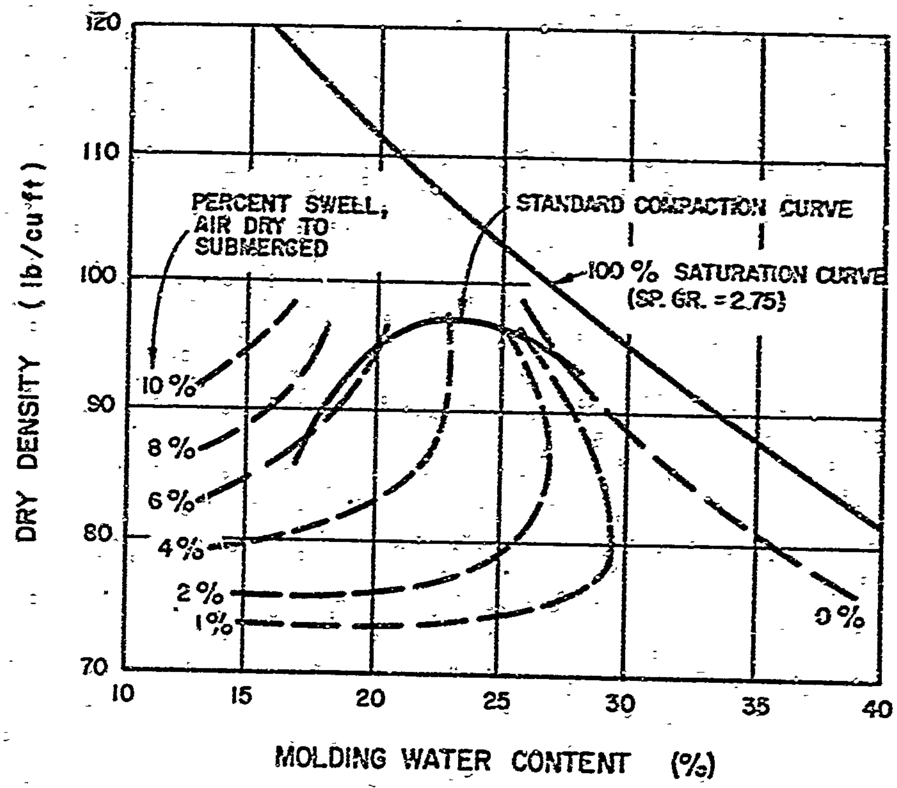


Figure 4. Percentage of Expansion for Various Molding Water Contents (After Holtz and Gibbs)

Applied external load Any force which tends to contain an expansive soil will also tend to restrain its volumetric expansion. The size of the load in relation to the swelling pressures developed will determine the magnitude of the swell.

One method of evaluating the swelling potential of an expansive soil is by determining the swelling pressure developed by testing the soil in such a manner that no volume change occurs. This swelling pressure is then the load required to prevent swelling from occurring.

Soil microstructure The clay microstructure may be a significant variable in influencing the magnitude of swell. The microstructure is largely a function of the type of clay minerals present, the exchangeable ions, the stress history of the deposit, and the environmental changes that occur (18).

The importance of particle arrangement has long been recognized as a contributing factor to engineering properties of soil. The early works of Terzaghi (19) and Casagrande (20) presented the first concepts of how these small particles were arranged and their influence upon the engineering properties of the soil. Lambe (21) presented an idealized representation of how the soil particles were arranged for an inorganic clay and the differences in microstructure of remolded clay (Fig. 5). He explained the basic differences in the structure using the concepts of colloidal chemistry. A flocculent microstructure is produced under a marine environment in which there is considerable "edge to face" contact between particles

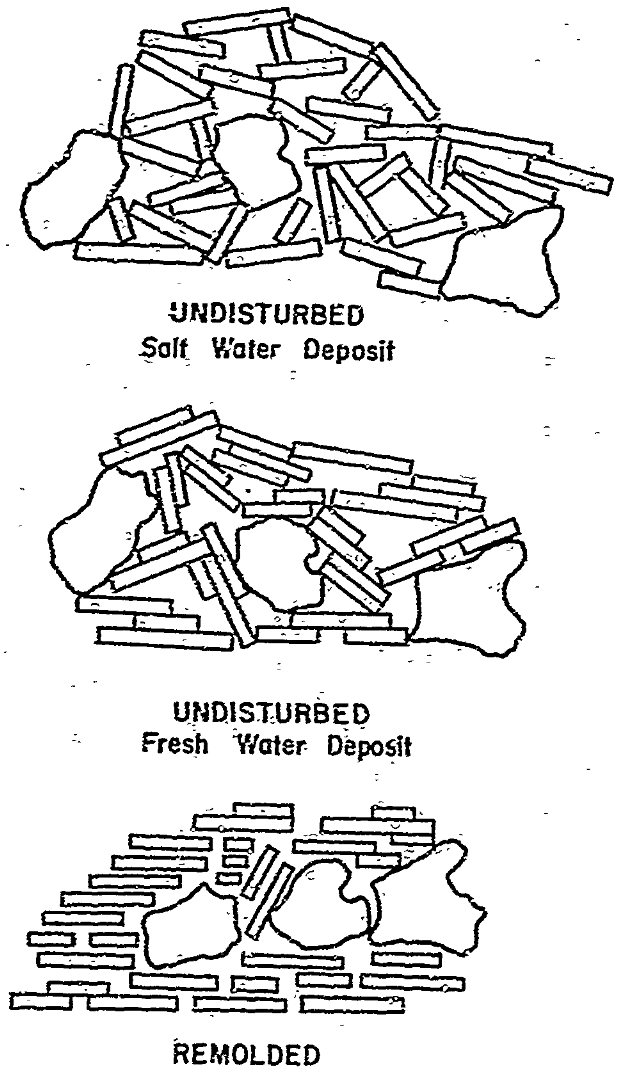


Figure 5. Particle Orientations in Clays (After Lambe)

whereas a fresh water microstructure will have a more "face to face" orientation or oriented microstructure. Whether or not actual contact between the mineral surfaces does occur is still a subject of discussion.

It seems reasonable to accept that some form of semi-solid contact is established either between mineral to mineral or through a few Angstroms of very strongly adsorbed water, with the latter being the more likely case.

The adsorbed water on the surface of the clay mineral has been shown by several investigators (22, 23, 24, 25, 26) to exist in a state much different from free water. While the thickness is a subject of disagreement, all agree that the water closest to the clay particle shows the greatest difference in its properties from that of free water, and is held to the clay particle by great forces. Bolt (22) showed with consolidation tests on clays having essentially parallel orientation that the average spacings between particles continued to decrease with pressures up to 100 atmospheres. This illustrates the extreme difficulty in extruding all the water from between the clay particles.

<u>Microstructure produced by compaction</u> The influence of the clay microstructure upon the engineering properties of a soil has been studied by several investigators by using compacted samples. Lambe (2, 13) in two excellent papers on compacted clay, has presented the concepts which have been generally accepted as to the nature of the microstructure produced by compaction at various

molded water contents. A flocculent microstructure is produced with water contents dry of optimum. This is a result of the incomplete water films around the clay particles which increases the tendency of the small particles to form flocs which are not broken down during compaction. As the water content increases the water films are increased and the particles can better align themselves (face to face) during the process of compaction. This effect is illustrated in Fig. 5.

The advantages of using compacted clay samples in studying microstructure are many. The primary advantage is that two different microstructures can be produced with a fairly uniform composition. Thus, the study of the microstructure is possible without the effect of layering which will occur during natural deposition.

Seed and Chan (27) found that the type of compaction may have a considerable influence upon the resulting microstructure when compacted wet of optimum moisture content. Samples compacted dry of optimum will normally retain their flocculent nature as the type of compaction will not destroy the orientation. They found that kneading and impact compaction appeared to produce a more dispersed structure wet of optimum. They reasoned that greater induced shear strains by these two types of compaction caused the particles to become oriented thus increasing the degree of "face to face" arrangements more than static or vibrating compaction.

Seed et al. (28) in a study of the swell and swell pressures of compacted clays, concluded that the microstructure was one of the

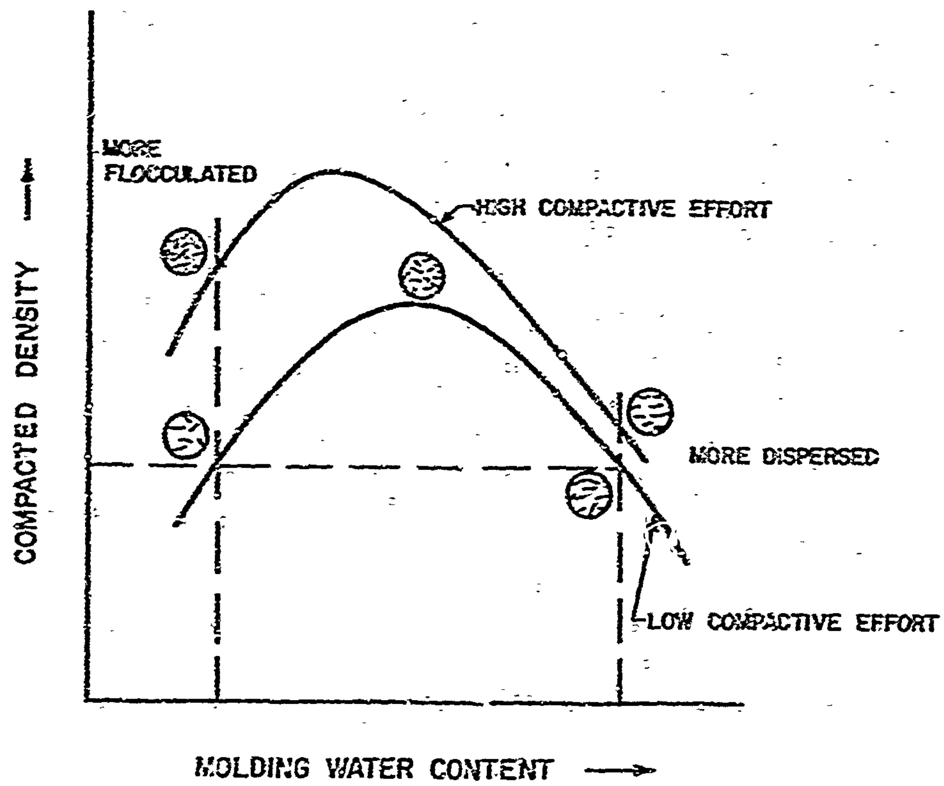


Figure 6. Effects of Molding Water Content on Microstructure (After Lambe)

major variables controlling the swelling behavior. Measurement of the microstructure was done indirectly based upon shrink-swell characteristics of the compacted samples. The more flocculent (compacted dry of optimum water content) microstructures had consistently higher swelling pressures and the largest volumetric increase.

Russam and Coleman (29) studied the compression and expansion characteristics of chemically prepared flocculent and parallel oriented microstructures. These results tend to support the earlier concepts of how the microstructure will influence the swelling behavior of clays.

There is little direct evidence of the microstructure produced by compaction. Aylmore and Quirk (30) studied the microstructure of clay by electron microscopes and advanced the concept of multigrain formations in which domains of oriented particles were arranged in a random manner or "turbostratic groups." Sloane and Kell (31) studied the microstructure of a pure kaolinite which had been compacted by several methods. They used the technique of Aylmore and Quirk (32) to prepare platinum-carbon replicas for electron microscope studies. The samples were air dried prior to study and the degree of disturbance to the microstructure by this process is unknown but could be significant. By studying thin sections under an electron microscope they observed that the microstructure was more of an oriented aggregate of particles or packets which was termed a "bookhouse" microstructure as being more descriptive

(Fig. 7). The degree of packet orientation increased with the molded water contents; however, the particle orientation in the packets did not appear to be significantly changed. They did observe various zones of orientation near the surface of the compaction hammer which extended as long chains of orientated packets forming shallow trajectories in the compacted clay mass. This supports the concept of Seed (27) that the shear strains induced in the soil during the compaction process increased particle orientation.

<u>Clay microstructure identification</u> The identification of the particle arrangements which exist in soil have been advanced significantly with the development of methods to evaluate the microstructure. The most common methods that have been used are X-ray diffraction, petrographic microscopy and transmission electron microscopy. The early work of Mitchell (16) using petrographic microscope techniques and the electron microscope studies by Rosenqvist (28) have generally confirmed the validity of the concepts of flocculent and oriented particle arrangements in natural sediments.

Use of the X-ray diffraction techniques for particle orientation has been done by Brindley (33, 34) and Martin (35). Their work was done to ensure complete randomness of clay particles so that X-ray diffraction patterns could be used as a quantitative method of determining amounts of the various clay minerals present.

Kaarsberg (36) used X-ray diffraction techniques to study the natural consolidation effect of particle orientation. He found that the particle orientation increased with depth for shales studied

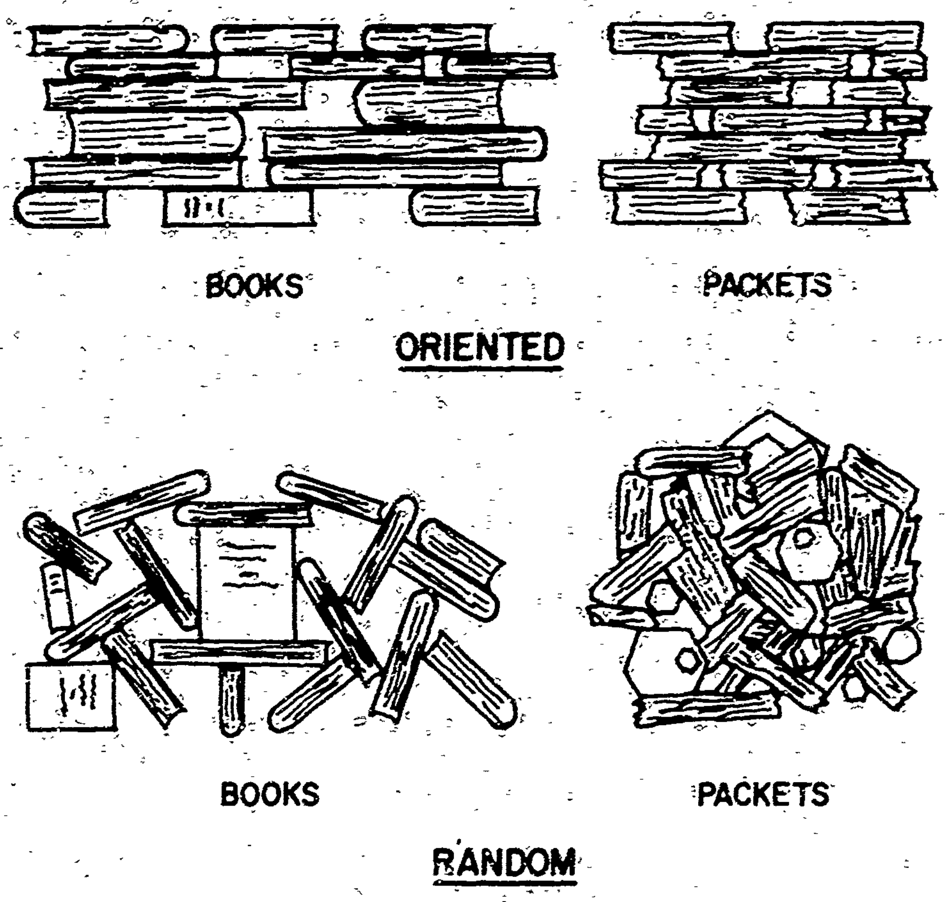


Figure 7. Book-Clay Packet Analogy (After Kell)

which were buried under 2700 feet or more of overburden. The method developed to compare orientations was based upon the ratio of (002) basal and (110) prismatic refraction spacings and bulk densities for samples.

Meade (37) used a similar technique in the study of preferred orientation in clays. In his analysis he used a ratio which compared the basal and prismatic peak intensities of a sample normal to bedding to those of a sample taken parallel to the bedding. This would tend to eliminate other factors which might affect the peak intensities for different samples such as particle size, chemical composition, and degree of crystallinity.

Other investigators (38, 39) have also successfully used X-ray diffraction techniques to determine particle orientation in natural sediments. These methods all involve relationships of basal and prismatic peak intensities. Odom (40) used a method involving only a comparison of basal intensities for parallel and normal samples. It is felt that this method must be open to criticism as the variability between samples previously mentioned may influence the results and interpretation of the degree of orientation.

Perhaps the most detailed visual evidence of particle orientation has been provided by transmission electron microscopy. The microstructure of marine deposits has received particular attention since the open structure can be easily studied by this method. The dense deposits provide some difficulty in interpretation as the particles are so closely packed. The largest drawback to this

method is in the sample preparation. The sample must be impregnated with some substance to replace the water prior to cutting ultra thin sections using diamond microtome knives. The problems associated with this complex preparation to prevent destruction of the existing microstructure have largely been resolved by Pusch (10) and Bowles (41). Both have used nitrogen sublimation to remove the water before impregnating the samples with various compounds to provide the required rigidity of the sample for thin section preparation.

A recent advancement in the field of electron microscopy has been the development of a Scanning Electron Microscope. Image formation, in the case of the Scanning Electron Microscope, differs from that of the conventional electron microscope and optical microscope, whose images are formed directly by lenses, in that the image is formed on a cathode ray tube after first converting information from the specimen's surface into a train of electrical signals. In the Scanning Electron Microscope, a finely focused electron probe scans the specimen under study by bombarding the specimen.

Data is then accumulated from many points to build up a representation of the area being viewed as the probe strikes only one point on the specimen at a time. At the same time, the cathode ray tube is scanned in synchronization with the electron beam so that each point on the cathode tube represents the same point on the area being viewed.

Generally, the Scanning Electron Microscope is characterized by a great depth of focus, high resolving power, simple preparation

of specimen and low intensity of incident electron probe.

The great depth of focus is a very desirable feature for microstructure work, as a larger range of particle sizes can be kept in focus. The other main advantage of the Scanning Electron Microscope is in sample preparation. Relatively large samples can be used, thus eliminating the need to use more sophisticated and time consuming techniques to prepare ultra thin sections required for the transmission electron microscopy work.

The use of Scanning Electron Microscopy is continually broadening into various fields. The use as a tool for microstructures has not been extensive. A recent paper by Borden and Sides (42) is perhaps the first application to soils engineering. They investigated the influence of microstructure on the collapse of compacted clay. They evaluated various techniques of sample preparation including numerous impregnation agents, methods of fracturing the sample and treatments of the fractured surface prior to coating with a conducting substance. Of interest is the fact that the final technique used was air drying the sample prior to fracturing the surfaces to be viewed. They concluded the air drying did not cause a significant distortion to the microstructure.

Summary

(a) Generally, it can be stated that the higher the density of the expansive soil the greater the swell that will occur.

(b) The magnitude of the external loading placed upon an expansive soil will influence the resulting swell. If the load

applied to the soil is equal to the swelling pressure developed, no swell will occur.

(c) The concepts of flocculent and parallel orientation of clay particles in natural clay deposits has generally been confirmed by various techniques including X-ray diffraction, electron microscopy and petrographic microscopy.

(d) The influence of the soil microstructure upon the engineering properties of a soil has been largely inferred from its physico-chemical properties.

(e) The microstructure in compacted clays is controlled by the molding water content and the compactive effort. A flocculent microstructure is produced on the dry side of optimum moisture content and the degree of orientation will improve as the molding water is increased.

(f) The recent concepts in microstructure evaluation are toward a multi-grain or packet formation in which the particle orientation is not changed by mechanical manipulation (compaction) but rather the packets themselves are oriented in either a flocculent or dispersed orientation.

(g) The type of compaction may influence the degree of orientation significantly on the wet side of optimum moisture content. Kneading and impact compaction may produce a higher degree of orientation than static compaction.

(h) The effect of particle orientation upon the swelling characteristics has been limited to indirect observations without an attempt to determine the actual microstructure by visual or other means.

SECTION IV

MATERIALS AND TEST PROCEDURES

A highly expansive clay soil which contained a significant amount of montmorillonite was selected for study to meet the stated objectives.

The soil used was a San Saba Clay obtained from Bosque County, Texas. This material was sampled from a cultivated field, 1.2 miles northwest of the intersection of State Highway 174 and Farm Road 927 near Morgan, Texas (Fig. 8). The material is dark gray, has a black-granular structure, and contains a limited amount of visible organic matter. The soil was sampled from a depth between 6-16 inches.

The San Saba Clay was then air dried, pulverized and the soil fraction passing the U. S. No. 40 sieve was utilized in the testing program. Prior to use, the clay was modified by the addition of five percent on a dry weight basis of a relatively pure highly crystalline kaolinite clay obtained commercially from Dresser Industries, Kosse, Texas. The addition of the kaolinite clay was required to enhance the study of the microstructure. The kaolinite was predominantly of clay size.

To assure that a uniform mixture of kaolinite and San Saba Clay was obtained and also that the kaolinite particles would become an integral part of the soil matrix, the following procedure was used. After combining the two soils in an air dry condition, water was added in a fine mist, and they were mixed by hand. The soil was then placed in double plastic bags and stored for a period of at least

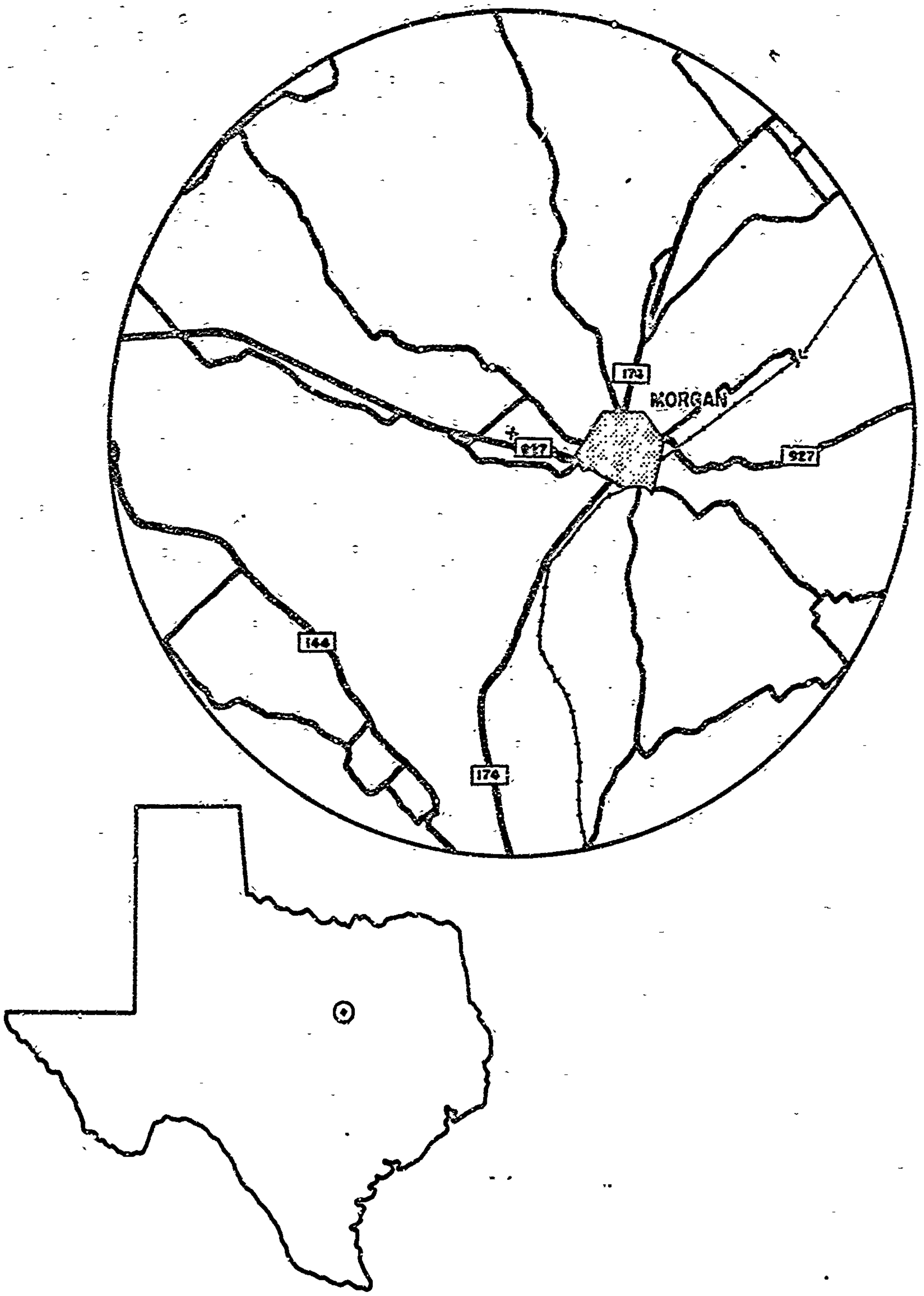


Figure 8. Location Map

seven days in the moist room. The soil was then compacted, broken up and allowed to dry at room temperature. After drying, the soil was again pulverized to pass the U. S. No. 40 sieve and the procedure repeated. The kaolinite lost its white appearance and could no longer be distinguished from the other soil particles. This indicated uniform soil matrix was obtained.

The soil mixture will henceforth be referred to as San Saba Clay.

Test Methods All laboratory testing was performed on the campus of Texas A&M University with the exception of the clay microstructure study using the Scanning Electron Microscope. The Scanning Electron Microscope used was provided by the Southwest Institute of Advanced Studies located in Richardson, Texas.

The facilities utilized at Texas A&M University included Department of Civil Engineering, Soils Mechanics Division and Materials Testing Division Laboratories, the Department of Agronomy and Plant Sciences, and Soil Physics and Clay Mineralogy Laboratory. All laboratory tests were performed by the author or under his direct supervision.

Engineering Properties Test Methods The following standard engineering tests were performed on the selected soil.

Specific gravity The specific gravity of the soil particles is an important physical property which is used in most mathematical relationships where volume or weight of the soil sample is being considered. The specific gravity of a soil is an average density of all soil particles present and is expressed as a ratio of the mass of soil particles to their volume, excluding pore spaces between particles.

In determination of the specific gravity for the soil, a Beckman Air Comparison Pycnometer was used. In essence, the air pycnometer utilizes the same principle as the standard liquid displacement method (43) except that air is used instead of deaired water. The pycnometer is calibrated directly so that any measured difference in the two chambers, one containing air, the other the soil sample, will provide the volume of soil. The particle density of specific gravity is then found knowing the dry weight and the volume occupied by the soil particles.

<u>Atterberg Limits</u> The Atterberg Limits tests were performed utilizing the standard equipment developed by Casagrande (44) and procedures as given by Lambe (43). Prior to testing, the soil samples were allowed to become fully saturated by remaining in distilled water for a minimum of 36 hours.

<u>Mechanical analysis</u> The particle size distribution of the whole soil was accomplished by sieve analysis to determine the sand size distribution while a hydrometer analysis was utilized to obtain the silt and clay particle size distributions. The standard procedures used are those given by Lambe (43). The clay particle distribution was further verified by fractionation of the 2μ - 0.2μ and <0.2μ clay particles during the course of the mineralogical analysis.

<u>Method of compaction</u> Kneading compaction was selected for this study since normally a greater variance of microstructure can be obtained on wet and dry side of optimum moisture contents. The soil was compacted with a California Kneading Compactor and the moisture content dry density relationship obtained for the soil.

The compactive effort used was 200 psi foot pressure, with a dwell time of 0.5 seconds. The soil was compacted in five equal layers with 25 tamps per layer, in a 1/30 cu ft standard "Proctor" compaction mold.

To increase the uniformity of the compacted specimen, it was desirable to limit the formation of lumps as the molding water was added. A technique of adding the molding water in a fine mist and hand mixing the soil was found to be successful in reducing the size and quantity of these lumps. After mixing, the soil was then placed in double plastic bags and stored in the moist room for a period of at least seven days.

<u>Compaction of swell test specimens</u> The moisture contents to be used for the swell test were selected from the dry density versus moisture content curve. Points of equal dry densities were desired in a range of 2-5% on either side of the optimum moisture content. The same procedure of compaction was used for preparing samples for the swell test. However, after mixing several batches at the same moisture content, the moist soil samples were combined into a large plastic bag. After curing for about three days, moisture content samples were taken and adjustments made if required. The soil was then returned to storage until the seven-day period was complete. Prior to the start of a swell test, the required amount of soil for one sample was withdrawn and the specimen compacted. The soil remaining in the large plastic bag was checked for moisture content periodically and no changes were observed. The compacted densities also remained constant.

Mineralogical Analysis Methods

<u>X-ray diffraction analysis</u> The mineralogical composition of a soil is an important factor in understanding its behavior. The X-ray diffraction analysis of both the San Saba Clay and Kaolinite Clay was performed using the techniques developed by Jackson (45) as modified by Dixon (46).

Pretreatment of the soil consisted of removal of soluble salts, carbonates and organic matter to facilitate fractionation of the various particle sizes. Fine sand particles were separated by wet sieving using a U. S. No. 270 sieve (0.053 mm). The fine and coarse silt particles were then removed by successive sedimentation using centrifuge techniques. The fractionation of the clay was also performed using centrifuge methods. The following fractions were obtained for X-ray diffraction analysis. 50-5μ, 5-2μ, 2μ-0.2μ, and <0.2μ. The clay fractions were then saturated with magnesium chloride and glycolated to optimize interpretation.

The X-ray diffraction patterns were obtained using a North American Phillips high angle goniometer model instrument using a scanning speed of 1 and 2 degrees per minute 2θ.

Soil Microstructure Analysis Methods

<u>X-ray diffraction method</u> The principle involved in the study of clay microstructure by X-ray diffraction is very simple. The intensity of the X-ray peak from a particular crystal plane is controlled by two factors:

(1) Clay concentration in the irradiated volume, and,

(2) Orientation of clay particles in that volume.

In other words, the diffractometer can only record reflections from those crystals which satisfy the Bragg equation and are situated in a plane parallel to the axis of rotation of the specimen.

The clay minerals present in the soil used in this investigation consist of mainly montmorillonite and kaolinite. Due to the different moisture contents to be used in establishing the microstructure, the montmorillonite did not have well defined X-ray peaks. Therefore, kaolinite was chosen as an indicator mineral for particle orientation.

The X-ray diffraction peaks selected for the microstructure interpretation were the basal reflection (002) and the prismatic reflection (020).

The Peak Ratio (PR) as developed by Martin (36) was used as one of the parameters used to compare particle orientation.

$$PR = \frac{(002)\ \text{amplitude}}{(020)\ \text{amplitude}}$$

The PR will indicate a change in particle orientation since as the clay particles become more orientated (parallel to the compacted effort), the (002) basal reflection will increase and the prismatic reflection (020) will decrease.

In order to minimize the variation between the compacted samples caused by the different dry densities tested (mineral concentration) a ratio of the peak ratios was used. The Orientation Ratio (OR) was established which is equal to

$$OR = \frac{PR_{\parallel}}{PR_{\perp}}$$

$PR_{\parallel}$ = PR of a sample parallel to the direction of compaction

$PR_{\perp}$ = PR of a sample normal to the direction of compaction

The OR will eliminate the differences between compacted samples caused by variations in mineral and particle size concentrations of the tested samples. This ratio will provide a valid means of comparing the degree of orientation for the selected values of dry densities from the compaction curve.

The OR was then determined for selected points on the dry density vs moisture content compaction curve. Theoretically, the limits of the OR would range from 1.0 for a random microstructure and approach zero for a completely parallel oriented sample.

The samples to be tested were cut from the center portion of the compacted specimen with adjacent samples being cut in a parallel and perpendicular plane to the compaction effort.

Initially, the specimens were cut approximately 1.0 x 1.8 inches and 0.4 inches thick. The cut specimens were then placed in a solution of Carbowax 6000 in a closed container which was maintained at 60°C (16). All samples were found to be completely impregnated after remaining about five days in the Carbowax. The specimens were then removed and allowed to cool to room temperature before final shaping. The samples were carefully ground using carborundum to an exact thickness of 0.2 inches and approximately 0.9 x 1.5 inches in length and width. Thus, the area which had been disturbed during sample

cutting prior to impregnation was removed. The samples in their final form had two large smooth parallel surfaces for X-ray diffraction. The specimens were then stored in a desiccator prior to testing.

Special specimen mounting holders were constructed of plexiglass to assure proper alignment in the diffractometer during running of the test. Various scanning speeds were tried and speeds of 1/2° and 1/4° per minute were selected as being the best for the desired data.

<u>Scanning Electron Microscope method</u> A JSM-1 Scanning Electron Microscope, manufactured by Japan Electron Optics Laboratory Company, Ltd. was used in this study. The mode of operation of the electron microscope used was secondary electron imagery as this mode provided the highest degree of resolution in the scanning microscope. Once the probe was properly focused on the specimen, the changing of magnification or specimen orientation did not require any change in focus. This is one of the reasons that the depth of field is enhanced and improves the interpretation of scanning electron micrographs. The depth of field is one of the most desirable features of the Scanning Electron Microscope.

The samples selected for study under the electron microscope were those of the two selected moisture contents which corresponded to the two states of microstructure.

Initially, small cubic specimens were obtained from the center of the compacted sample. The specimens were then fractured both vertically and horizontally to give approximately 0.3 inch cubic specimens. The cubes were then trimmed and cut diagonally to give a prismatic specimen with a base dimension of about 0.25 inches. Two orthogonal planes which had not been disturbed by cutting which represented a surface parallel and one normal to the direction of compaction were then prepared for study.

The fracture surfaces were cleared of the layers of damaged and reoriented particles caused by the fracturing process by a method described by Borden and Sides (42) using a number of applications of adhesive tape.

After the surfaces were cleaned, the specimen was mounted on a special specimen plug and coated with gold palladium by metal evaporation to render the surface electrically conductive for viewing in the Scanning Electron Microscope. Several other coating metals were used, however, gold palladium provided the best results. Samples were then placed in the specimen chamber of the microscope and viewed at various magnifications from 600 to 8000 times. Photomicrographs were taken of selected representative areas and enlarged by photographic means.

Swell Test Apparatus and Methods

The Bishop Oedometer The Bishop Oedometer, HCl Hydraulically Pressurised Consolidation Cell manufactured by WYKEHAM FARRANCE ENGINEERING LIMITED, was originally developed as an improvement to the standard method of running the consolidation test. A schematic diagram is presented showing the basic components of the oedometer in Fig. 9. The oedometer with top removed is shown by Fig. 10.

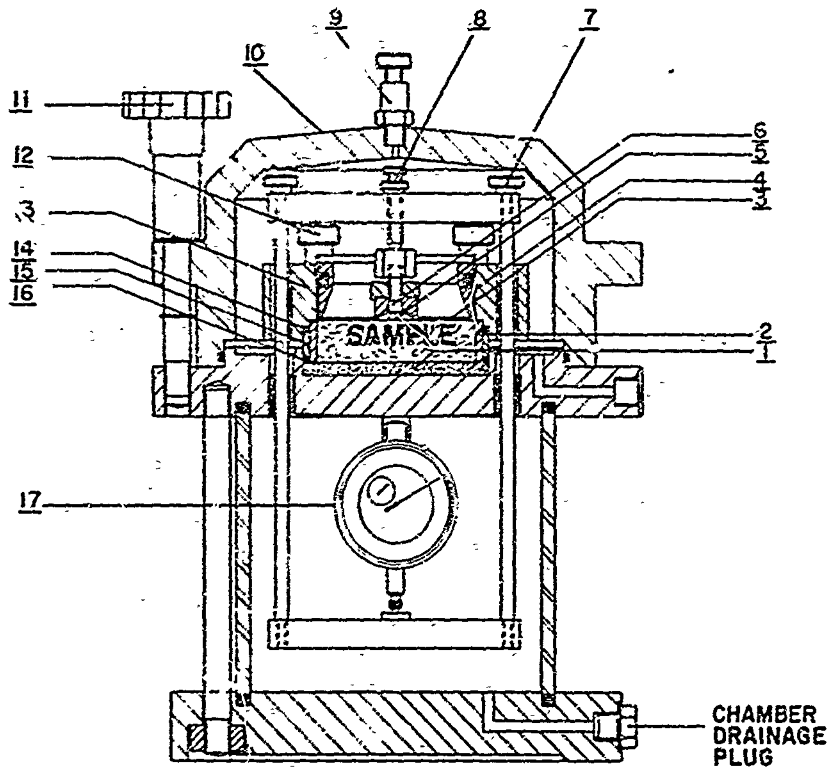
The advantages of this type of consolidometer are:

(1) It is possible to measure pore pressure.

(2) It utilizes a flexible piston to transmit the hydraulic load to the upper surface of the soil.

(3) It reduces the effects of the side friction since the measuring pad is located in the center of the sample.

The only modification necessary to the oedometer, in order to measure high suction pressures, is to replace the standard porous stone with a high air entry ceramic stone. This was done, using a 5 Bar ceramic stone made by the SoilMoisture Equipment Corporation, Santa Barbara, California for this study. The stone was then cemented into the bottom of the chanber using a high strength epoxy glue (E-POX-E glue No. EPX-1, manufactured by Woodhill Chemical Company, Cleveland, Ohio) which had the desirable property of expanding slightly when submerged in water for any period of time.



KEY

1. HIGH AIR ENTRY CERAMIC STONE
2. SAMPLE RING
3. FLEXIBLE RUBBER MEMBRANE
4. TAPERED INTERNAL RING
5. POROUS STONE
6. SMALL METAL MEASURING AREA
7. LOADING YOKE NUTS
8. LOADING YOKE ADJUSTMENT SCREW
9. CHAMBER PRESSURE RELEASE VALVE
10. CHAMBER TOP
11. CHAMBER CLAMPING SCREWS
12. FOUR LOCKING NUTS
13. LOCKING RING
14. UPPER "O" RING
15. SAMPLE RING CONTAINER
16. LOWER "O" RING
17. DIAL GAGE

Figure 9. Bishop Hydraulic Oedometer

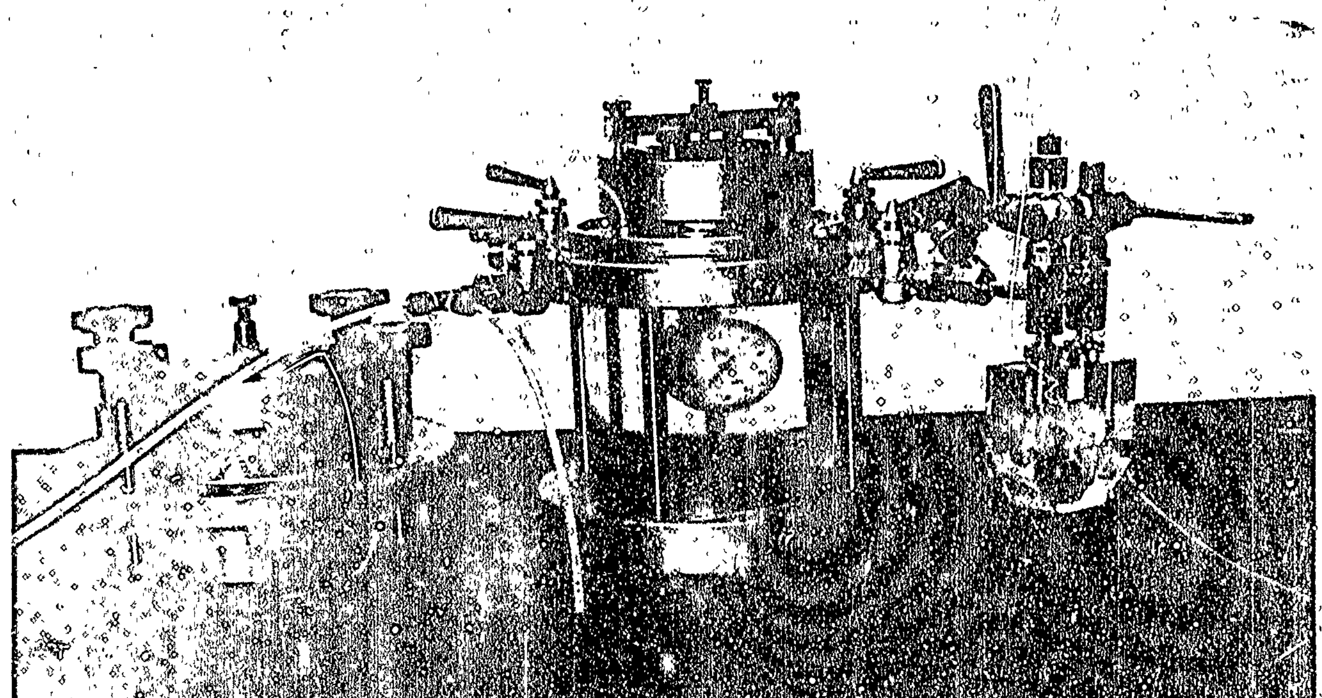


Figure 10. Bishop Oedometer with Null Gage Attached

Once the stone was cemented into place it was saturated by filling the chamber with demineralized, deaired water and allowing the systems to remain under high pressure for approximately 24 hours. About 100 ml of water was then forced through the stone to remove any air which was trapped in the stone and had gone into solution under the high pressure. The chamber was then depressurized, drained and filled with air. The air bubbling pressure of the stone was checked. It was found that pressures of 75 psi could be maintained without air bubbling through the stone and a failure of the measuring system.

Supporting system In order to meet the objectives set forth, the following supporting system was designed to provide the control and capabilities desired. A schematic diagram of the entire system is given in Fig. 11.

Air pressure chamber To provide the necessary air pressures for the axis of translation technique it was required that constant air pressures could be maintained for lengthy periods and that these pressures could be accurately measured.

To meet these requirements it was decided to use an air-water pressure chamber which was connected to an air supply through a 100 psi pressure regulator which could maintain constant pressures. The water in the pressure chamber was connected to the central valve system through a long length of tubing, approximately 15 feet. This was done to prevent contamination of the deaired water in the rest of the system when pressure measurements were taken.

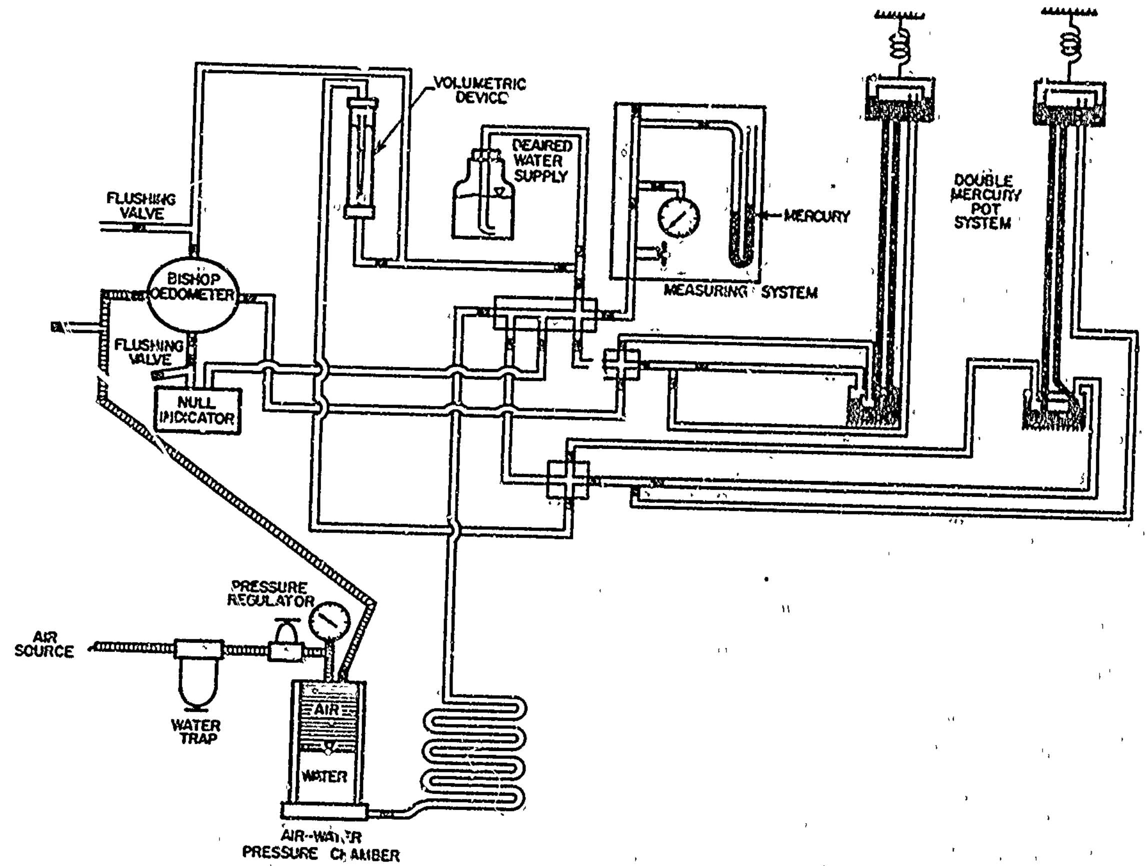


Figure 11. Schematic Diagram of Testing System

Load and back pressure system The use of self-adjusting mercury pots to provide constant pressures for long periods of time is well established. If any small leak develops or a change in volume in the sample, the self-adjusting spring on the pots will compensate and maintain the pressure at a constant value.

Two sets of self-adjusting mercury pots were used. One was utilized to apply the loading on the specimen while the other was used to apply water at known pressures to the sample. The pressures were transmitted through the central valve and measurement systems to the Bishop Oedometer.

Measuring system The measuring system used was similar to that given by Bishop and Henkel (47). The main components consisted of a pressure gage, calibrated in 0.10 kg/cm^2, a mercury manometer calibrated in 0.01 kg/cm^2 with a maximum reading of 1.8 kg/cm^2 and a screw control pump for fine adjustment. This system was used to make all pressure measurements for the entire system.

The manometer was adjusted to give a zero reading for water level at the top of the ceramic stone.

All components of the system were interconnected through a central valve network and manifold which increased the ease of operation and maintained a better control over the entire operation.

Pore water pressure measurement apparatus Pore water pressures were measured by using a Bishop null indicator, illustrated in Fig. 12. The principal advantage of the null indicator is that the pore pressure can be balanced rapidly by applying an equal pres-

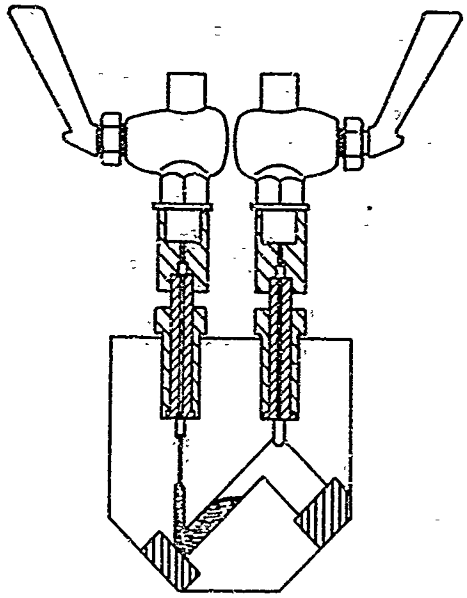


Figure 12. Bishop Null Indicator

sure to the other side of the mercury column preventing any flow of water either into or out of the soil sample. The small diameter (1/16 inch) mercury column provides high sensitivity in measurements, whereas, the larger bore tube provides an adequate volume of mercury to prevent loss of the mercury column due to accidentally unbalancing the pressures.

For flushing and deairing the apparatus, a mercury trap is provided so that free passage of water is possible by tilting the null indicator and allowing the mercury to flow into the trap.

Prior to testing, the mercury column is raised to the desired level and the system balanced. By increasing the pressure rapidly, the null indicator can be checked for any trapped air bubbles which are indicated by a change in the mercury water interface. Movement of less than 1/16 inch was considered to be a result of elastic expansion of the null indicator and not a result of trapped air. After the initial deairing of the null indicator, the operation only required flushing with fresh deaired water prior to starting a new test.

<u>Moisture content measurement</u> A Bishop volume change device was used to measure the amount of water that flowed into the sample during an application of back pressure. This device, Fig. 13, measures the volume change by the displacement of the surface between water and colored kerosene. The inner tube is calibrated so that the volume of water displaced may be read directly. The apparatus was used in conjunction with the self-compensating mercury pots which

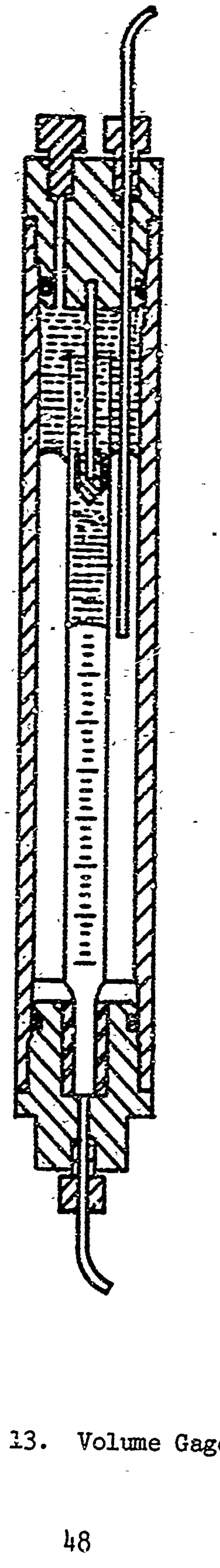


Figure 13. Volume Gage

applied the required back pressure. The change in moisture content could be calculated knowing the volume of water flowing into the sample, initial moisture content and density of the specimen.

Swell test procedures Before beginning any test the entire system was flushed with demineralized deaired water and the ceramic stone saturated.

The compacted soil sample was cut and trimmed to size in the cutting mold, and the ends trimmed flush to the sample holder. Finished size of the sample was 3 inches in diameter and 0.75 of an inch thick. The sample was then mounted in the oedometer.

Once the sample was placed in contact with the ceramic stone the oedometer was assembled as rapidly as possible. The upper chamber was filled with water by using a separate pressure chamber to supply water rapidly to the chamber. In order to prevent cavitation during the assembling and filling the upper chamber it was necessary to open the valve on the pore pressure system for a fraction of a second to relieve the negative pressures. This procedure was used by Olson and Langfelder (5) and the small amount of water moving into the sample should not affect the soil suctions significantly.

Once the upper chamber was filled with water, the air pressure and chamber water pressure were increased simultaneously in the axis translation technique. The dial gage was then read and this reading taken as the initial value. The loading was then applied to the specimen by use of the mercury pot system. The pressure applied was equal to the desired loading plus the applied air pressure.

The sample was then allowed to come to equilibrium with the applied load. Periodic measurements of the pore water pressures and the dial gage were taken. Equilibrium was reached when the pore water pressure remained constant over a period of time and no further movement of the dial gage occurred. This point was normally reached in a period of time ranging from 5 to 24 hours.

The soil suction, which is the algebraic difference between the applied air pressure and measured pore water pressure, was then reduced in increments until the soil suction reached zero. This was done by applying a back pressure equal to the pore air pressure through the volumetric device to the oedometer. Periodic measurements of the amount of water moving into the soil were made until the desired quantity had penetrated the soil. The times of back pressure application depended upon the degree to which the suction pressure was reduced. They ranged from a few minutes to about 36 hours for the last increment which reduced the suction to zero. Periodic readings of the dial gage were taken. The swell was computed based on the percent vertical change from the original sample height.

After testing, the sample was removed and the moisture content checked. Prior to testing a new sample, the entire system was flushed with fresh, deaired, demineralized water.

SECTION V

RESULTS AND DISCUSSION

Mineralogy The mineralogical analysis of the San Saba Clay as modified by the addition of the kaolinite clay is presented in this section. The mineralogy of the kaolinite is given in Appendix I and will not be discussed in this section other than to say it is a relatively pure kaolinite of very high crystallinity.

Analysis of the mineralogy for the San Saba Clay was performed for the following soil fractions: >50μ, 50-5μ, 5-2μ, 2-0.2μ and <0.2μ. The X-ray diffraction patterns obtained for each fraction are contained in Appendix I. The fine silt (5-2μ) and clay fractions of the soil were saturated with magnesium and ethylene glycol to enhance the identification of montmorillonite. Table 1 summarizes the mineralogy of each of the soil fractions.

Discussion of the mineralogical properties The use of X-ray diffraction patterns is well established for the identification of the various clay minerals present in a soil. A mineral has certain diagnostic X-ray peaks which are a result of its crystalline structure. The location of these peaks has been well established by many investigators and can be used to determine the type of clay minerals present in a soil as well as a rough estimate of the amount of each.

The sand and coarse silt fractions of this soil consisted almost entirely of quartz. The diagnostic X-ray peaks for quartz are very strong and sharp peaks at 3.34 and 4.26Å. The fine silt (5-2μ)

TABLE 1 ESTIMATED ABUNDANCE OF CLAY MINERALS

Grain size	Minerals present
Sand, >50μ	Q_1[a]
Silt, 50-2μ	Q_1K_2
Coarse clay, 2-0.2μ	$K_2I_2M_3Q_3$
Fine clay, <0.2μ	$M_1K_2I_3$

[a] 1 - >40%
2 - 10-40%
3 - <10%

K - Kaolinite
I - Illite
M - Montmorillonite
Q - Quartz

fraction contains quartz, kaolinite and illite.

Kaolinite has X-ray peaks at 7.14 and 3.57Å whereas the illite was identified by its characteristic X-ray peaks at 10 and 5Å.

The clay was separated into coarse (2-0.2μ) and fine (<0.2μ) fractions. The coarse fraction contains mainly kaolinite and illite. A small amount of quartz was also present. The fine clay fraction consists largely of montmorillonite. The strong, sharp X-ray peak at 17.7Å is indicative that a large quantity of this mineral is present.

In summarizing the mineralogy of the San Saba Clay, the soil is composed of quartz, montmorillonite, kaolinite and a small amount of illite.

The presence of montmorillonite in a large quantity is a good indication that the soil has an expansive potential. Montmorillonite has the unique property of an expanding lattice structure, which

greatly increases its ability to adsorb large quantities of water. Also, the normally small crystals (<0.2μ), provide a large surface area for adsorption of water molecules per unit volume, further increasing its expansive potential.

San Saba Clay, due to its montmorillonite content, could be expected to be a somewhat troublesome soil on which to construct structures with light foundations. During seasonal fluctuation of moisture content, considerable shrinking and swelling may occur in the soil.

Engineering Index Properties The results of the standard engineering index properties are presented in Table 2 for the San Saba Clay. Included are the specific gravity, the liquid limit, plastic limit and plasticity index.

TABLE 2 ENGINEERING INDEX PROPERTIES

Specific gravity	2.63
Liquid limit	58.0
Plastic limit	21.6
Plasticity index	36.4
Grain Size Distribution (%)	
Sand, >50μ	5.6
Silt, 50-2μ	43.1
Clay, <2μ	51.3

The results of the compaction test are included in this section. The soil was compacted using the California Kneading Compactor. The optimum moisture content was 22.5% with a maximum dry density of 100.2 lbs/cu ft. The dry density-moisture content curve is given in Fig. 14.

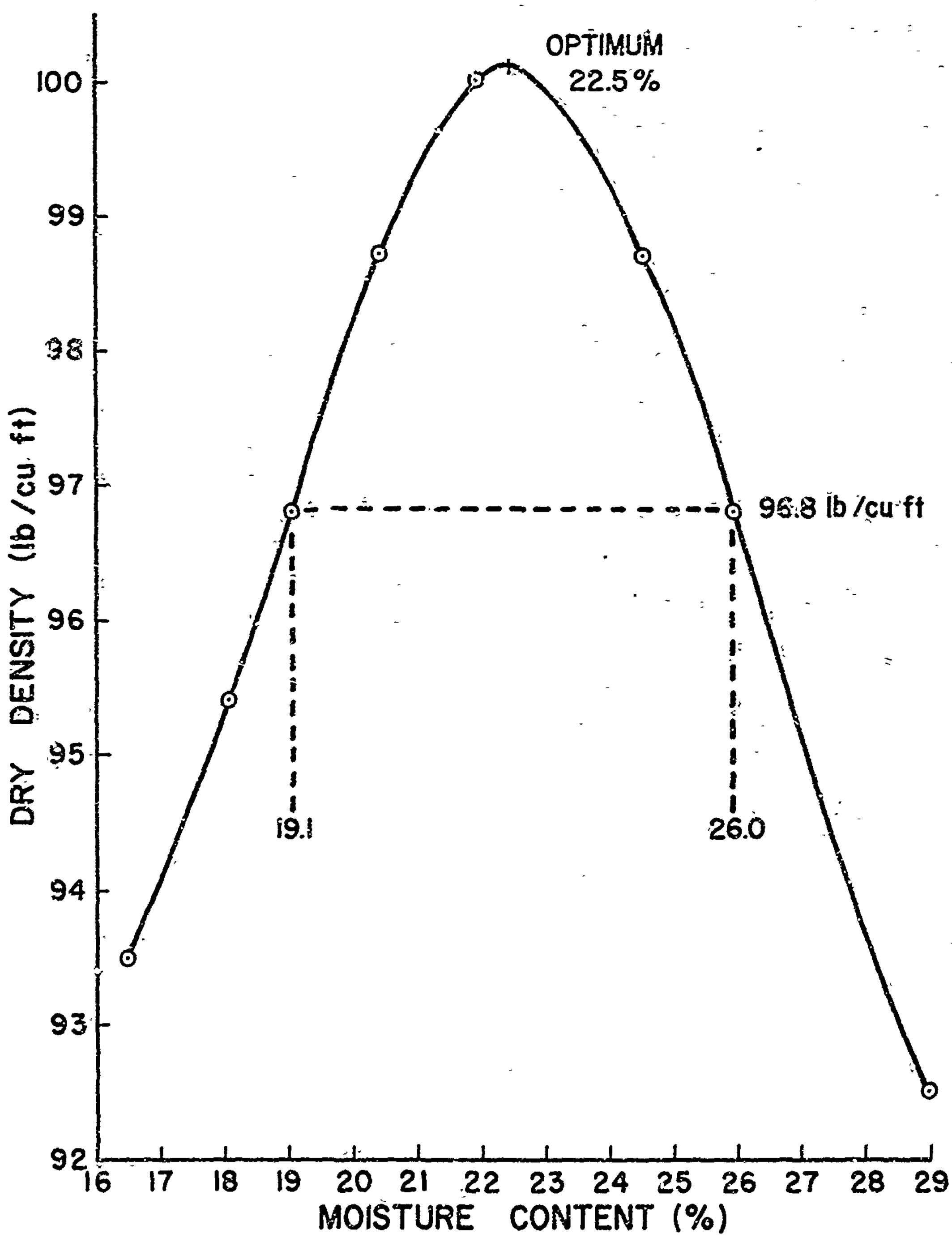


Figure 14. Kneading Compaction Curve for San Saba Clay

Based on the index properties and grain size distribution of the San Saba Clay, the soil would be classified as an inorganic clay of high plasticity - "CH" in the Unified Soil Classification System.

Discussion of engineering index properties The liquid limit of 58 is indicative that the clay has a considerable amount of montmorillonite, as was shown in the mineralogical analysis. In general, soils which have a Plasticity Index (I_p) greater than 30 will normally have a considerable shrink-swell potential. The I_p of 36.4 would indicate a moderate swelling potential for this soil.

The moisture content-dry density curve obtained from the kneading compaction was analyzed to select the molded moisture contents to be used for the swell test.

The moisture contents between 2-5% on either side of optimum were selected as the general range which would provide the desired microstructure changes and still keep the soil suctions compatible with the air entry range of the ceramic stone used. The dry density value of 96.8 lbs/cu ft was selected as meeting these criteria. Moisture contents of 19.1% and 26% were used. Sample preparation was developed so that the dry density could be duplicated throughout the testing period. A variance of ±0.3 lbs/cu ft was accepted as allowable.

The minimum curing period of seven days provided a constant density and no changes to compacted densities were observed at longer periods of curing.

Microstructure The data obtained from the study of the clay

microstructure is divided into two areas and discussed separately. These areas are the results of the (1) X-ray diffraction technique and (2) the Scanning Electron Microscope.

<u>X-ray diffraction results for microstructure</u> The X-ray peak intensities used for the determination of particle orientation included the (002) and (020) peaks of kaolinite. Typical results are given in Fig. 15 and the corresponding peaks identified with a "K" beside the respective number to indicate kaolinite. The data showing peak intensities are given in Appendix II. Fig. 16 shows the change of particle orientation of the compacted samples with changing compacted moisture contents. The maximum degree of dispersion is apparently reached near the optimum moisture content and decreases slightly with moisture contents significantly wet of optimum.

<u>Discussion of X-ray microstructure results</u> Kaolinite as an internal standard for the microstructure was selected for several reasons. First of all, the montmorillonite X-ray peaks were very diffuse and could not be easily distinguished. This is a result of several factors, the primary one being the incomplete saturation of the soil, which would cause a non-uniform lattice expansion of all the montmorillonite particles. This factor was very evident in the drier compacted samples as a very broad band was produced.

The second factor was the very weak prismatic peaks of montmorillonite. The high degree of background scatter in the X-ray patterns tends to obscure these peaks altogether.

Kaolinite, being the second most common clay mineral in the

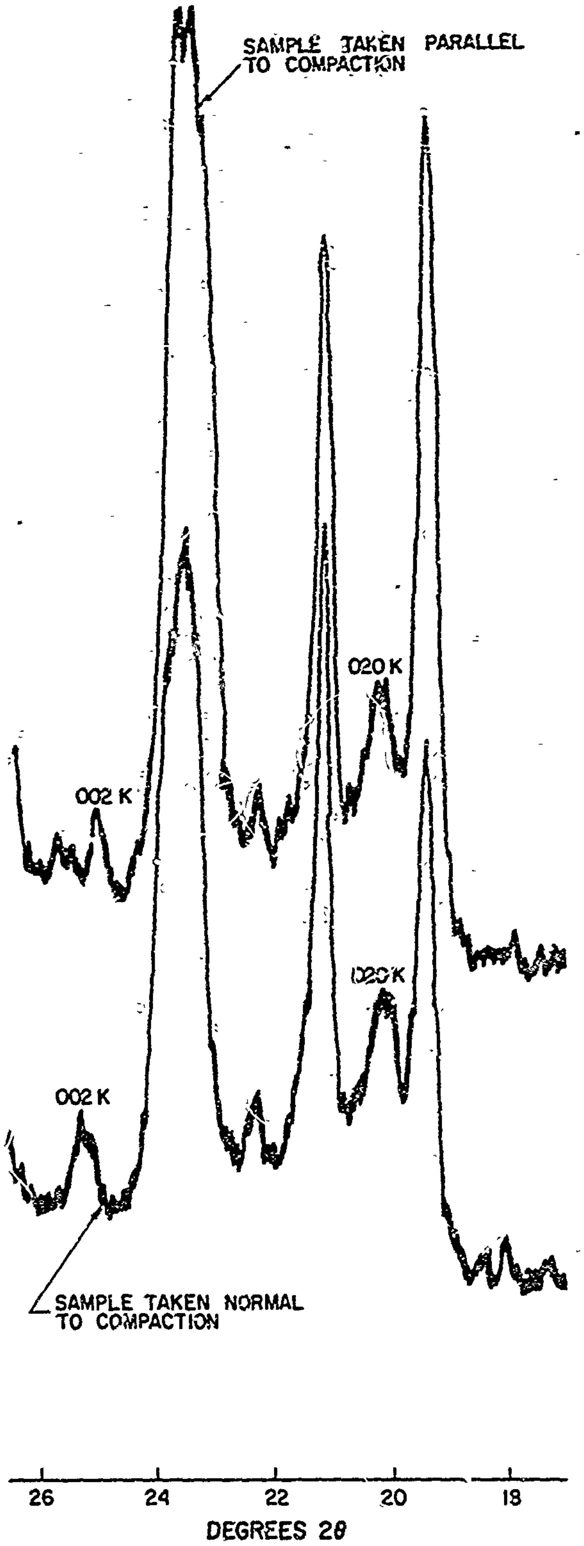


Figure 15. Typical X-Ray Diffraction Patterns of Microstructure Samples

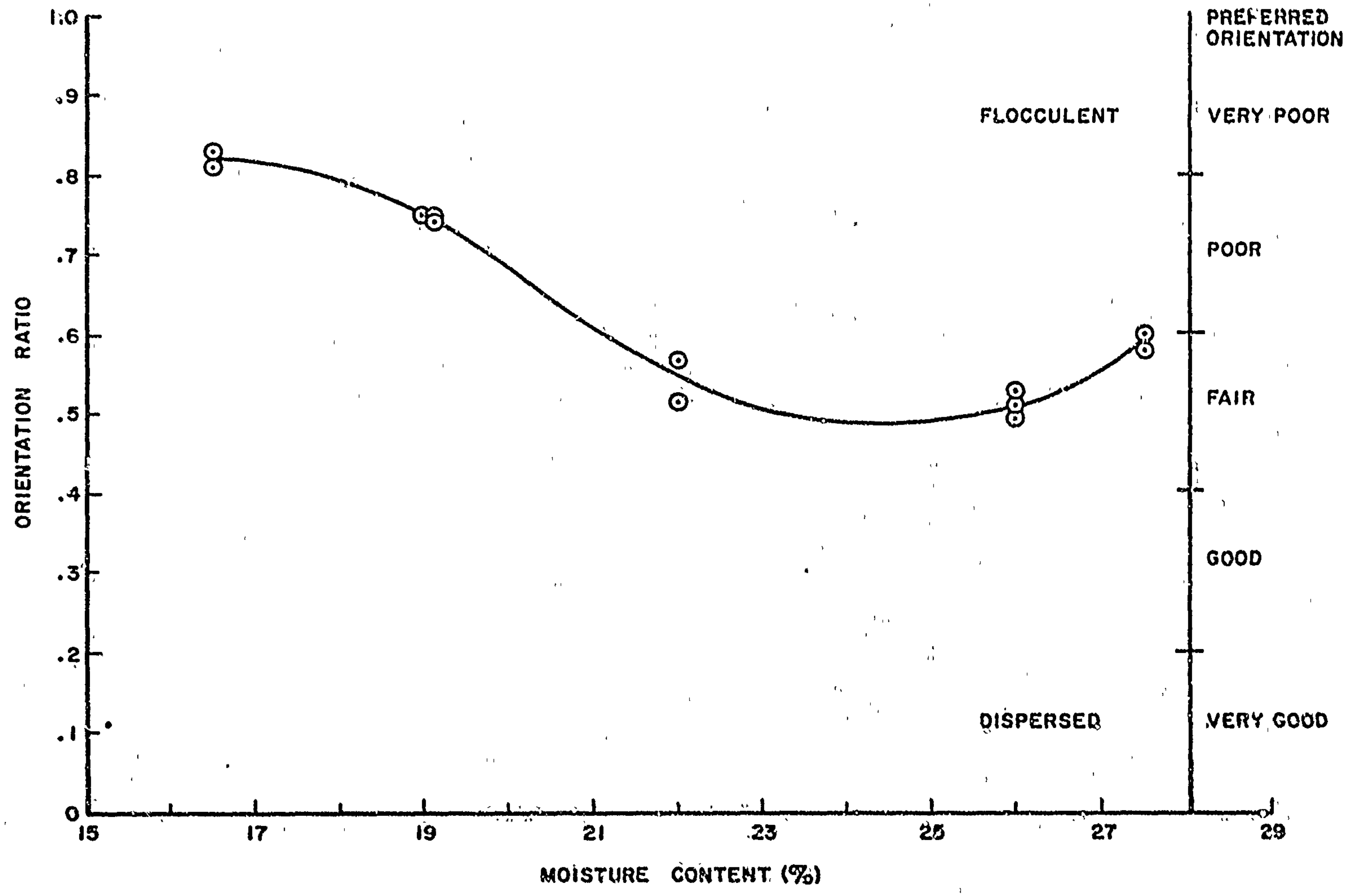


Figure 16. Orientation Ratio versus Moisture Content

soil matrix, was selected for microstructure analysis. Kaolinite has very strong distinct X-ray peaks which permits identification of the selected peaks possible.

From Fig. 16 it can be seen that dispersed orientation increases with increasing moisture contents until the optimum moisture content is reached. It then remains essentially the same until about 26% at which point apparently a decrease in preferred orientation begins. The decrease in dispersion may be explained by analyzing what happens during compaction as the soil approaches saturation. For particle orientation to change, a stress must act on the soil particles. As the degree of saturation increases the stress transmitted to the particles during compaction is less as the water takes part of the stress. There would, therefore, be less force acting on the particles and consequently the rearrangement also would be less.

The orientation study of compacted clay by Lambe (2) using the petrographic microscope shows a definite change in slope of the orientation curve near the optimum moisture content. The degree of saturation for the various data points is not given but must be one of the factors influencing this change in slope. Cementing agents (organics, carbonates, etc.) present in the soil will also have an influence on particle orientation. The degree of reorientation possible by mechanical means may be somewhat limited if these bonds are strong.

The results of the X-ray analysis point out that the occurrence of multi-grain structures (packets) is very probable. However, it

is limited in defining whether packet or individual particle orientations are present in a soil as no visual evidence is provided. The X-ray analysis of the microstructure will give a good average orientation of the particles and will show overall gross orientation.

<u>Scanning Electron Microscope results for microstructure</u> In order to try to define whether packet or individual particle orientation was prevalent in the compacted samples, electron micrographs were taken of samples at the two test moisture contents used in the swell test. Both samples had the same dry density but were compacted at 19.1% and 26% molding water contents.

Representative electron micrographs are presented on the following pages. Surfaces normal and parallel to the compactive effort are shown.

<u>Microstructure at 19.1% moisture content</u> Figs. 17 and 18 were taken of the surface normal to the direction of the compactive effort. The general appearance of these figures is characterized by a multitude of packets which appear to be generally arranged in the plane normal to the compactive effort. Also, they give the impression of having a very open network of particle packets and relative large pores between the packets.

In Figs. 19 and 20 a surface which is parallel to the direction of the compactive effort is shown. These figures show a number of packet edges to be present. However, there is still an overall randomness of the orientation of these edges as well as some packet faces apparent in the micrographs (Fig. 20). Generally these

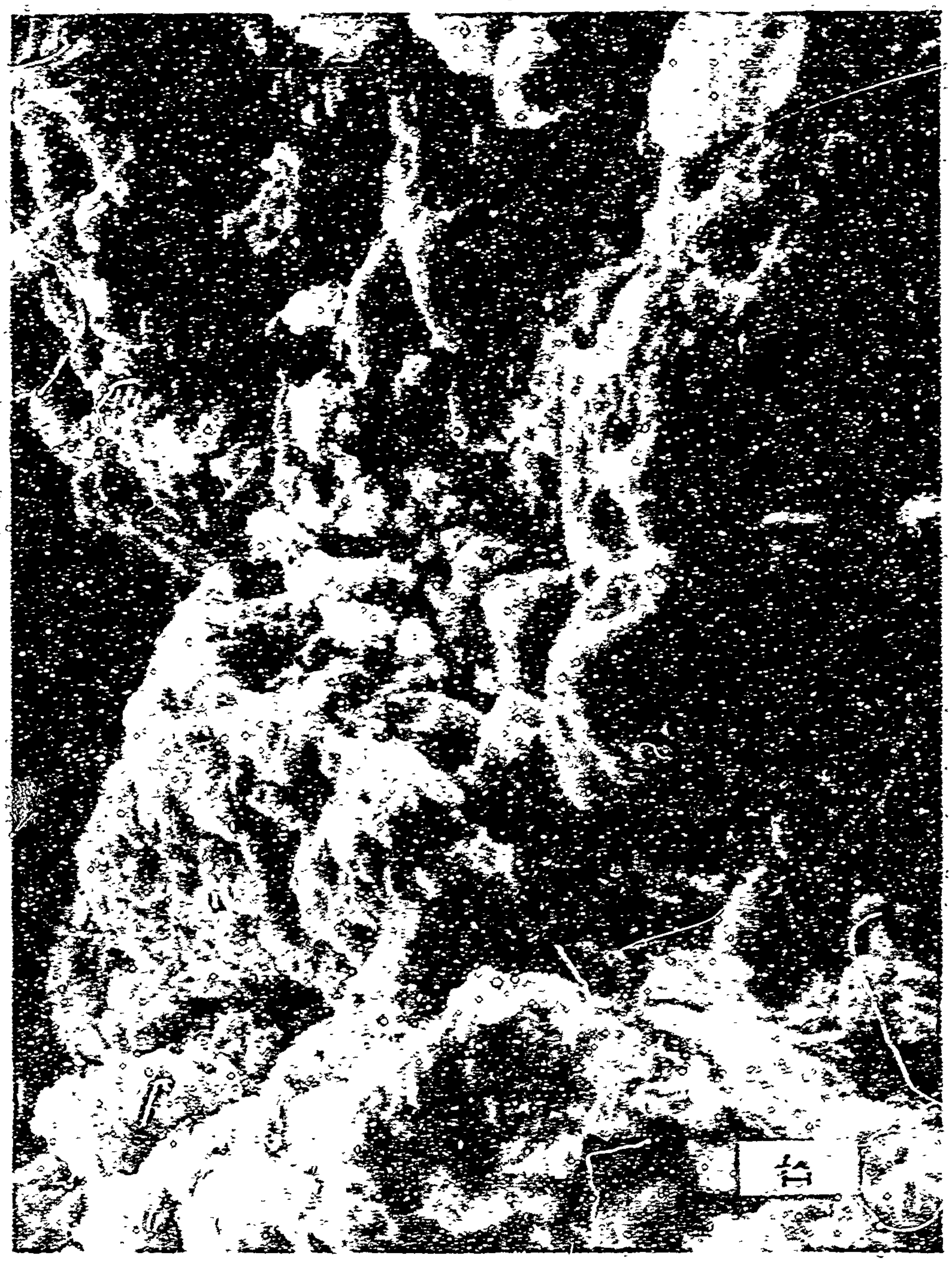


Figure 17. Scanning Electron Micrograph, View 1, Showing a Surface Normal to the Direction of Compaction (19.1%)

Figure 18. Scanning Electron Micrograph, View 2, Showing a Surface Normal to the Direction of Compaction (19.1%)

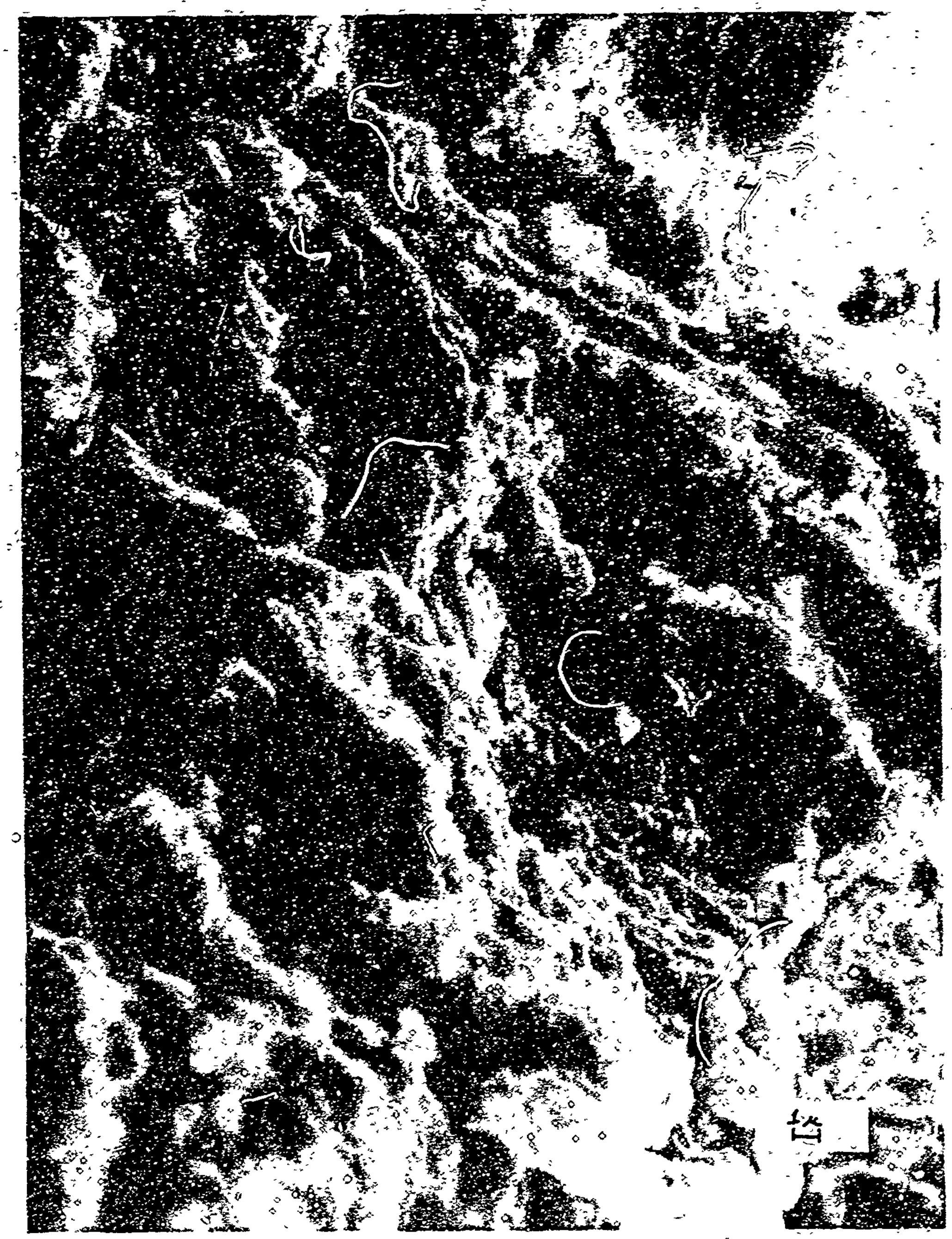


Figure 19. Scanning Electron Micrograph, View 1, Showing a Surface Parallel to the Direction of Compaction (19.1%)

Figure 20. Scanning Electron Micrograph, View 2,
Showing a Surface Parallel to the Direction of Compaction (19.1%)

micrographs give the appearance of a somewhat random orientation of packets and do not seem to be characterized by individual particle orientation.

Microstructure at 26% moisture content Figs. 21 through 23 were taken of the surface normal to the direction of the compactive effort and Figs. 24 and 25 represent a surface parallel to the compaction. In general, these micrographs appear to show a microstructure which is more massive and has a lesser degree of openness than the samples compacted at the drier moisture content. A high number of packet faces are shown normal to the direction of compaction but there is still a certain amount of randomness present. The micrographs showing a surface parallel to compaction still have a somewhat random appearance. The lesser degree of openness tends to give an impression that a higher number of packet edges are present. In the general topographic view (Fig. 24), this randomness can be seen.

Comparison of wet and dry side microstructure The microstructure illustrated in the preceeding micrographs is a much more complex system than the current idealized representations in the literature.

The surface normal to the compactive effort in both samples appears to have a large number of packets in a plane parallel to the micrograph. By comparing Figs. 18 and 21 the massiveness of the wet side sample is obvious as is the lack of large voids.

In comparing the surfaces parallel to compaction of the two

Figure 21. Scanning Electron Micrograph, View 1, Showing a Surface Normal to the Direction of Compaction (26%)

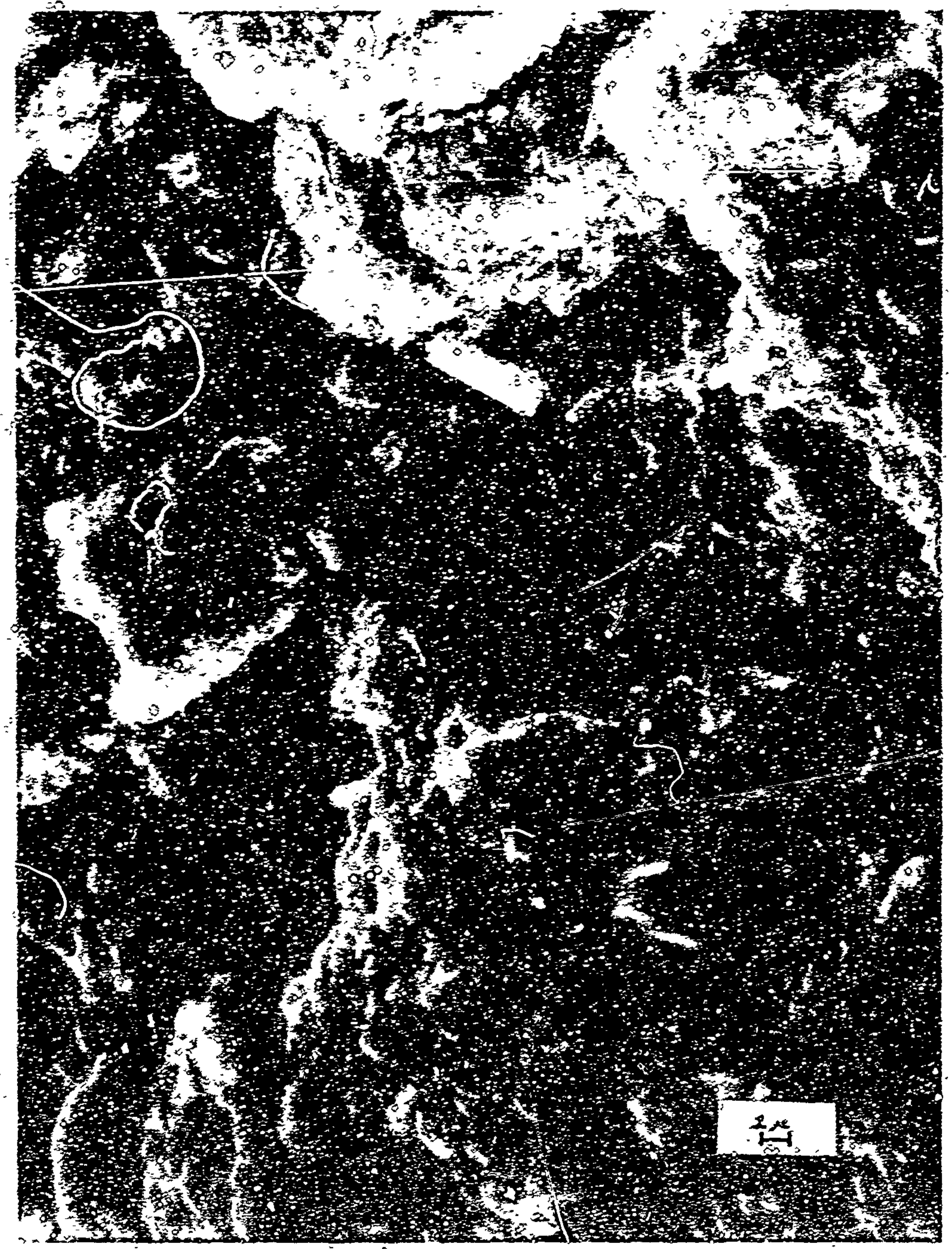


Figure 22. Scanning Electron Micrograph, View 2, Showing a Surface Normal to the Direction of Compaction (26%)

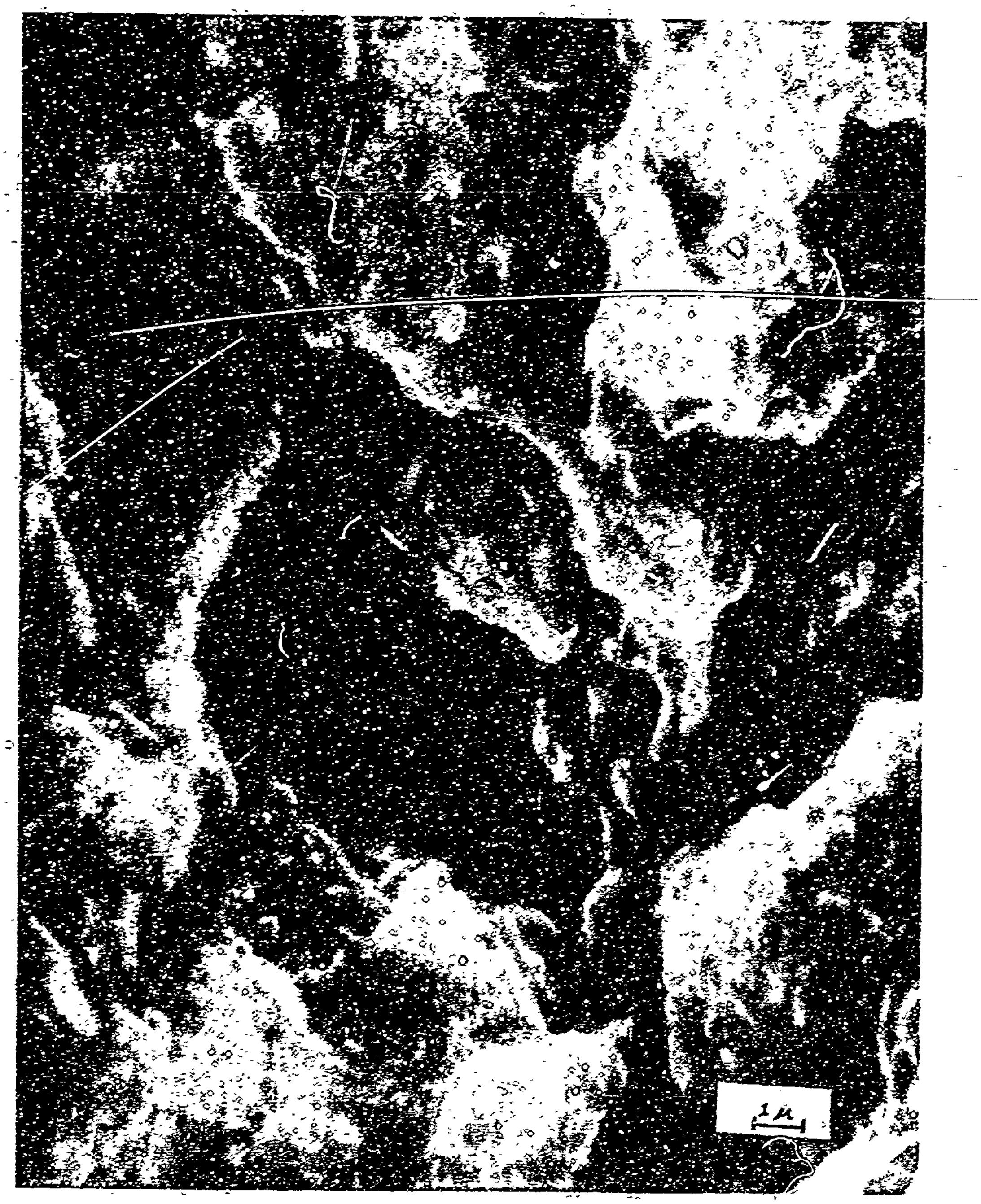


Figure 23. Scanning Electron Micrograph, View 3, Showing a Surface Normal to the Direction of Compaction (26%)

Figure 24. Scanning Electron Micrograph, View 1, Showing a Surface Parallel to the Direction of Compaction (26%)

Figure 25. Scanning Electron Micrograph, View 2,
Showing a Surface Parallel to the Direction of Compaction (26%)

moisture contents, there is a higher occurrence of "edges" in the wet side sample. This would indicate more particle orientation in this sample. However, the sample does not approach the degree of dispersion previously indicated in the literature.

The concepts of Sloane and Kell (31) are perhaps in closer agreement with the micrographs obtained. They concluded that the compacted soil consisted of multi-grain packets which will be in various degrees of orientation depending upon the molding water content for compaction. With standard electron microscopy, it has been very difficult to obtain a depth of field which would give some insight as to the actual make-up of the packets. In Fig. 18, the height of the packets is quite easily seen; however, it was not possible to distinguish the actual orientation of particles within the packets.

<u>Summary</u> The data obtained from both the X-ray diffraction and scanning electron microscopy techniques suggest that the change in the microstructure produced by kneading compaction (wet and dry of optimum) is not significant. For the soil tested, the maximum dispersion possible under the compactive effort used is reached around optimum moisture content and decreases slightly for moisture contents above 26%.

The electron microscopy results indicate that a multi-grain or packet formation is prevalent rather than individual particle orientation. Compaction was effective in causing increasing dispersion of the packets but it is felt that the orientation within the packets themselves was not altered significantly. Diamond (48) performed a

similar study on commercial kaolinite and illite and also concluded that the orientation for wet and dry side compaction was not significantly different using impact compaction.

Using the classifications of orientation of Odom (40), the microstructure would vary from a "poor" preferred orientation to one with only a "fair" degree of preferred orientation. While a certain amount of preferred orientation does exist in the wet of optimum sample, it does not approach a dispersed microstructure.

Swell Test Analysis This section is divided into three parts: results of the initial soil suction for the selected moisture contents to be used are presented first; the swell data for the wet and dry side compaction are presented separately. Values of suction are reported in units of kg/cm^2 rather than units of pF for ease of interpretation as the data was in a small range of pF.

Initial soil suction measurement The initial values of the soil suction for both the samples compacted at 19.1% and 26.0% moisture contents were determined. For this initial determination, the exposed end plate method of Gibbs and Coffey (11) was used.

Two different procedures of applying the air pressure required for axis translation were used. In one test, the air pressure was increased gradually in small increments to keep the measured pore water pressure slightly positive. In another test, the total required air pressure to prevent cavitation from occurring was applied at the start of the test.

Figs. 26 and 27 show the equilibrium time and soil suction

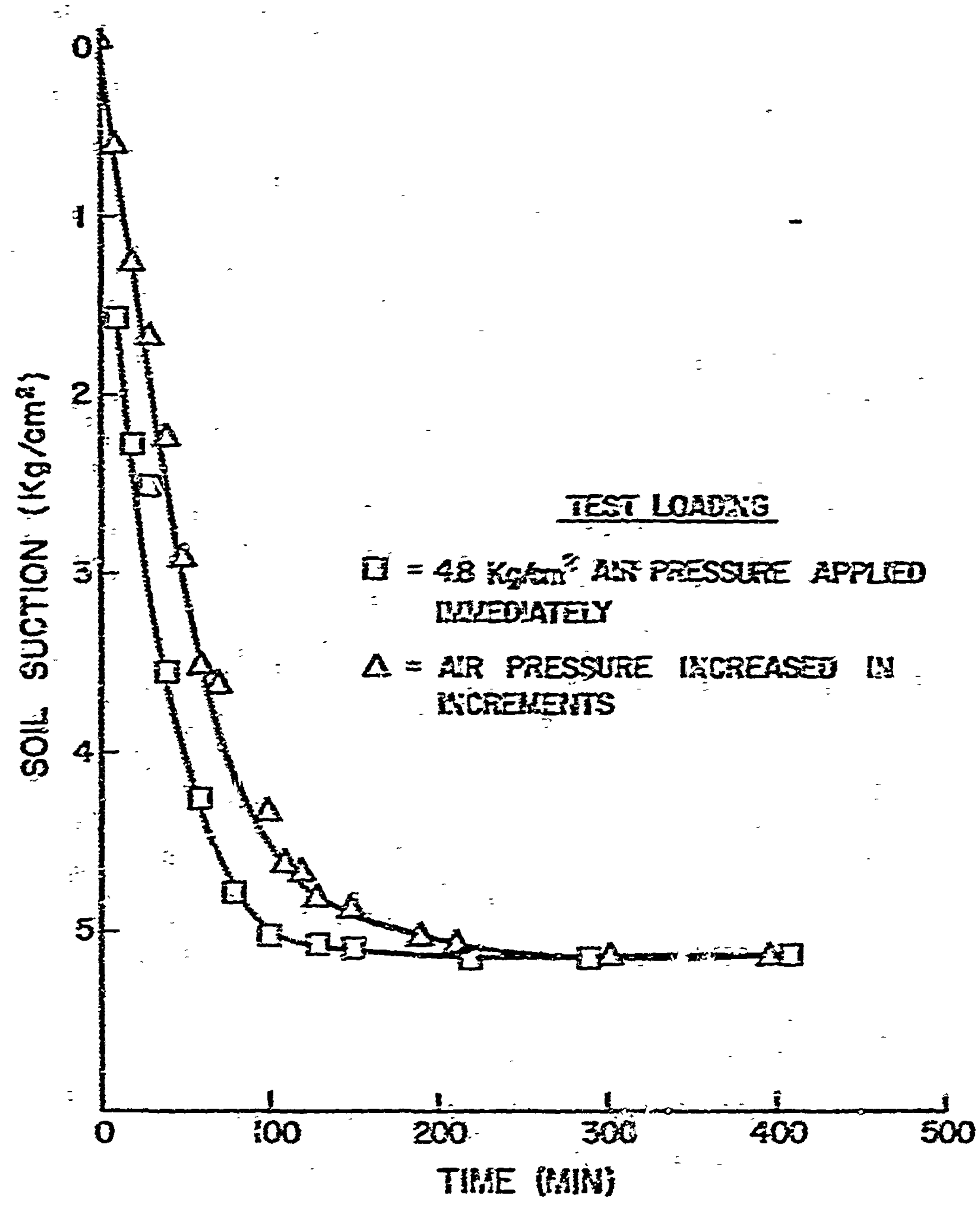


Figure 26. Soil Suction versus Time of Applied Air Pressure for Initial Moisture Content of 19.1%

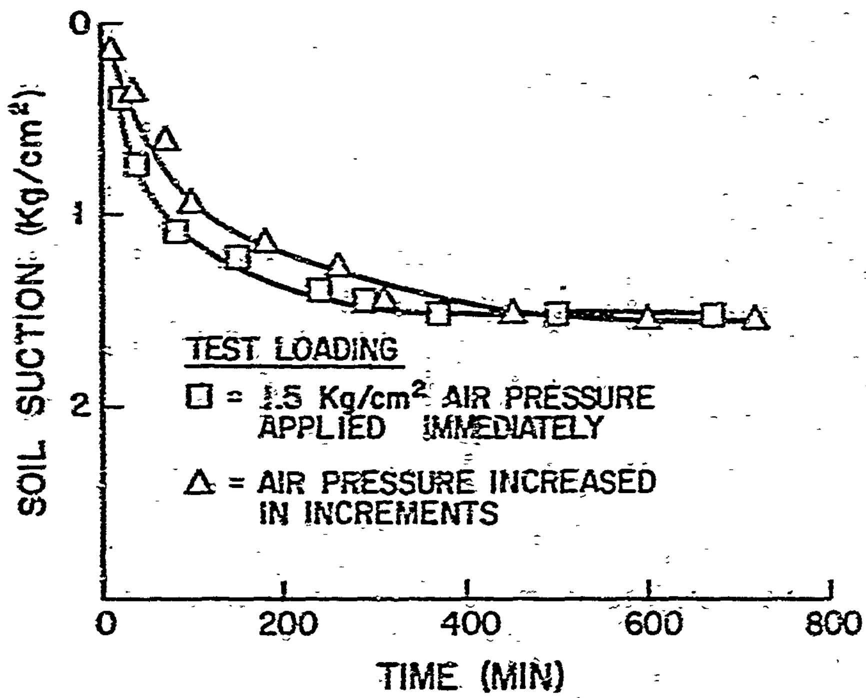


Figure 27. Soil Suction versus Time of Applied Air Pressure for Initial Moisture Content of 26%

relationships. The initial suction for the 19.1% moisture content sample was 5.18 kg/cm^2 for the gradual increase of applied air pressure and 5.14 kg/cm^2 for an initial air pressure application of 4.8 kg/cm^2. The 26% moisture content sample had a value of 1.52 kg/cm^2 at an air pressure of 1.5 kg/cm^2, whereas by gradually increasing the applied air pressures, an initial suction of 1.55 kg/cm^2 was measured.

Discussion of initial soil suction measurement The determination of the initial soil suction values for the compacted samples was very important for several reasons. Primarily, it was necessary to determine if the capability of the ceramic stone was adequate. If the soil suction were greater than the air entry value of the stone, then cavitation would occur. The pore pressure measurement system would then break down and inaccurate data would be obtained. In some early tests, when cavitation did occur, the measured pore water pressures began to increase rapidly and with time approached values of the applied air pressure.

The time required to establish equilibrium between the ceramic stone, applied air pressure and the soil sample was greatly influenced by the initial condition of the ceramic stone. If the stone was too wet, a considerable lag time resulted before equilibrium was reached. It was found that by lightly wiping the surface of the stone with clean filter paper the proper initial condition of the stone could be obtained. Equilibrium times ranged from 4 to 6 hours. It was also observed that by increasing the contact area between the

stone and soil to about 85%, the time required for equilibrium was decreased. Initially the tests were performed using about 70% contact area as suggested for the exposed end plate technique but resulted in equilibrium times exceeding 8 hours.

A comparison between the two methods of applying the required air pressure was necessary to determine if any significant differences occurred in the initial value of soil suction. In the swell test, the air pressure would be applied at the start of the test. The slight differences of 0.04 kg/cm^2 and 0.03 kg/cm^2 were quite acceptable and were probably caused by small differences in samples themsleves.

<u>Swell test results for samples compacted at 19.1%</u> The relationship between soil suction and moisture content is shown in Fig. 28. The influence of the applied external loading is demonstrated by the family of curves which started at a suction of about 3 kg/cm^2. The increase in loading had two effects. First, the point at which the curve left the straight line part of the suction vs moisture content curve was at progressively lower soil suctions as the load was increased. Second, the final moisture contents were reduced as the load increased and varied from 30.2% at a zero loading to 24.2% at a load of 1.5 kg/cm^2.

Fig. 29 shows the influence upon the swell of both the decreasing soil suction and the applied loadings. Maximum swell under zero load was 6.9% and decreased to 0.44% at 1.5 kg/cm^2. As the size of the test loads increased, consolidation occurred during the

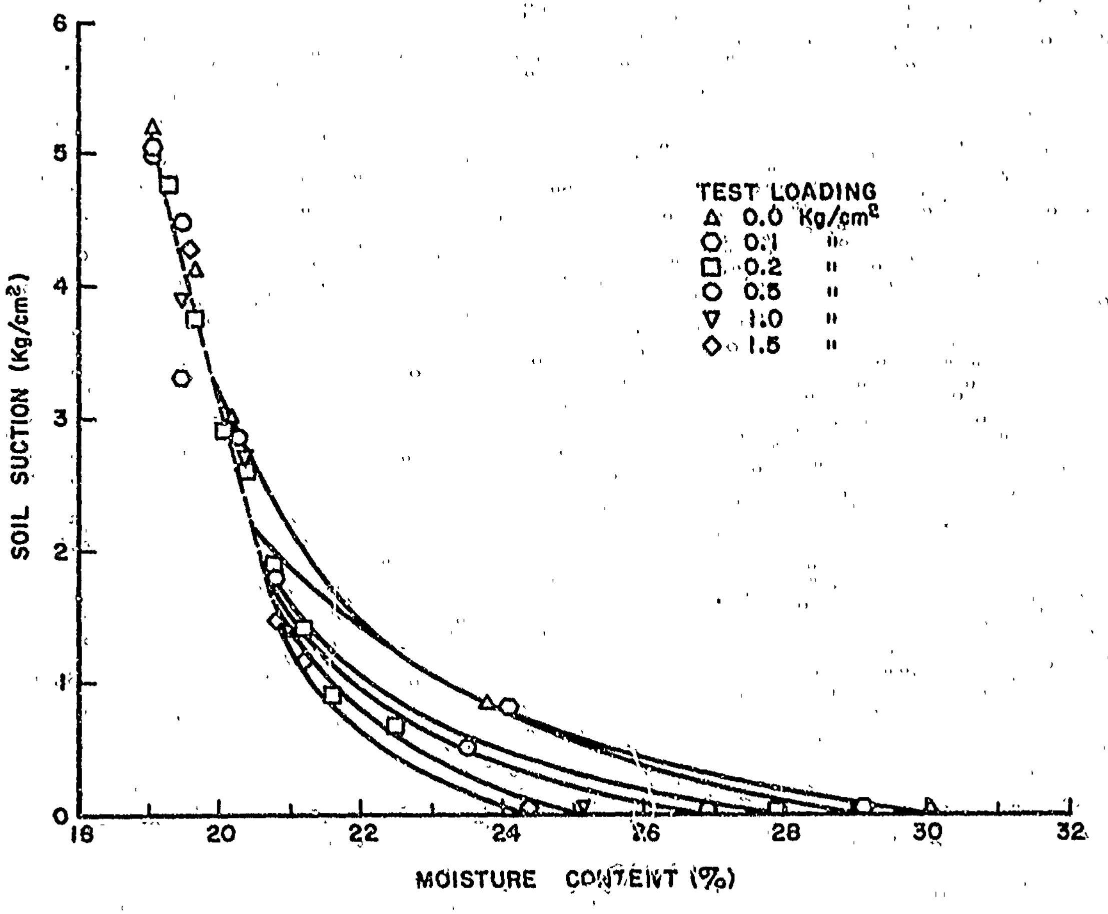


Figure 28. Effect of Increasing Moisture Content on Soil Suction for Initial Moisture Content of 19.1%

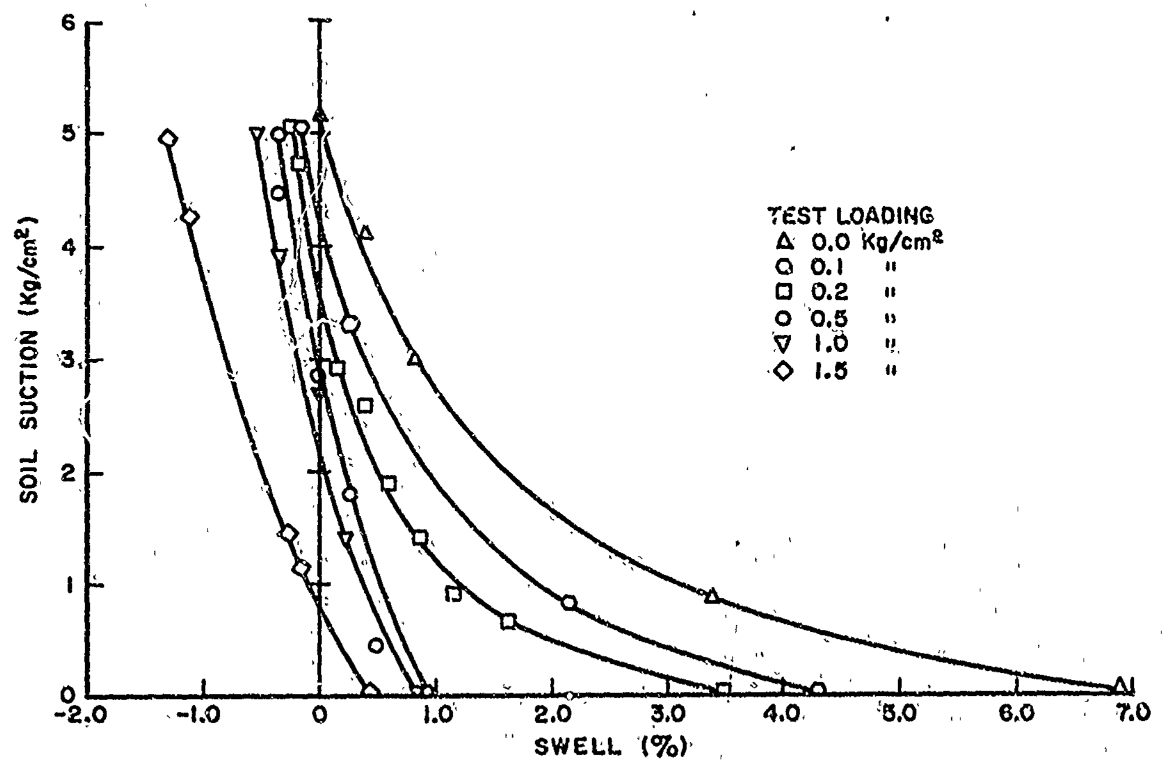


Figure 29. Effect of Decreasing Soil Suction on Swell for Initial Moisture Content of 19.1%

equilibrium period. The consolidation appeared to have very little effect upon the initial soil suction.

Fig. 30 gives the relationship between swell and moisture content for the various test loadings.

The experimental data is given in Appendix III. Table 3 provides a tabular summary of load, swell, moisture content values and degree of saturation.

TABLE 3 SWELL DATA SUMMARY 19.1% INITIAL MOISTURE CONTENT

Applied loading, kg/cm^2	0.0	0.1	0.2	0.5	1.0	1.5
Initial soil suction, kg/cm^2	5.18	5.04	5.04	5.00	4.98	4.96
Initial degree of saturation, (%)	72.3	72.6	72.7	72.3	73.2	74.7
Initial moisture content, (%)	19.1	19.1	19.1	19.1	19.1	19.1
Final moisture content, (%)	30.2	30.0	27.9	26.9	25.1	24.4
Final degree of saturation, (%)	97.9	99.8	97.2	99.5	93.2	91.3
Swell, (%)	6.9	4.7	3.5	0.96	0.85	0.44

Discussion of results for swell tests (19.1%) It would appear from examining Fig. 28 that the test loading had very little effect upon the initial soil suction of the sample. As the moisture content increased, the corresponding reduction of the soil suction followed the same approximate path to a suction of about 3.0 kg/cm^2. Then, depending upon the loading, the actual point of departure was at a lower value of suction with increased loading.

To examine the possible reasons for this, it is necessary to

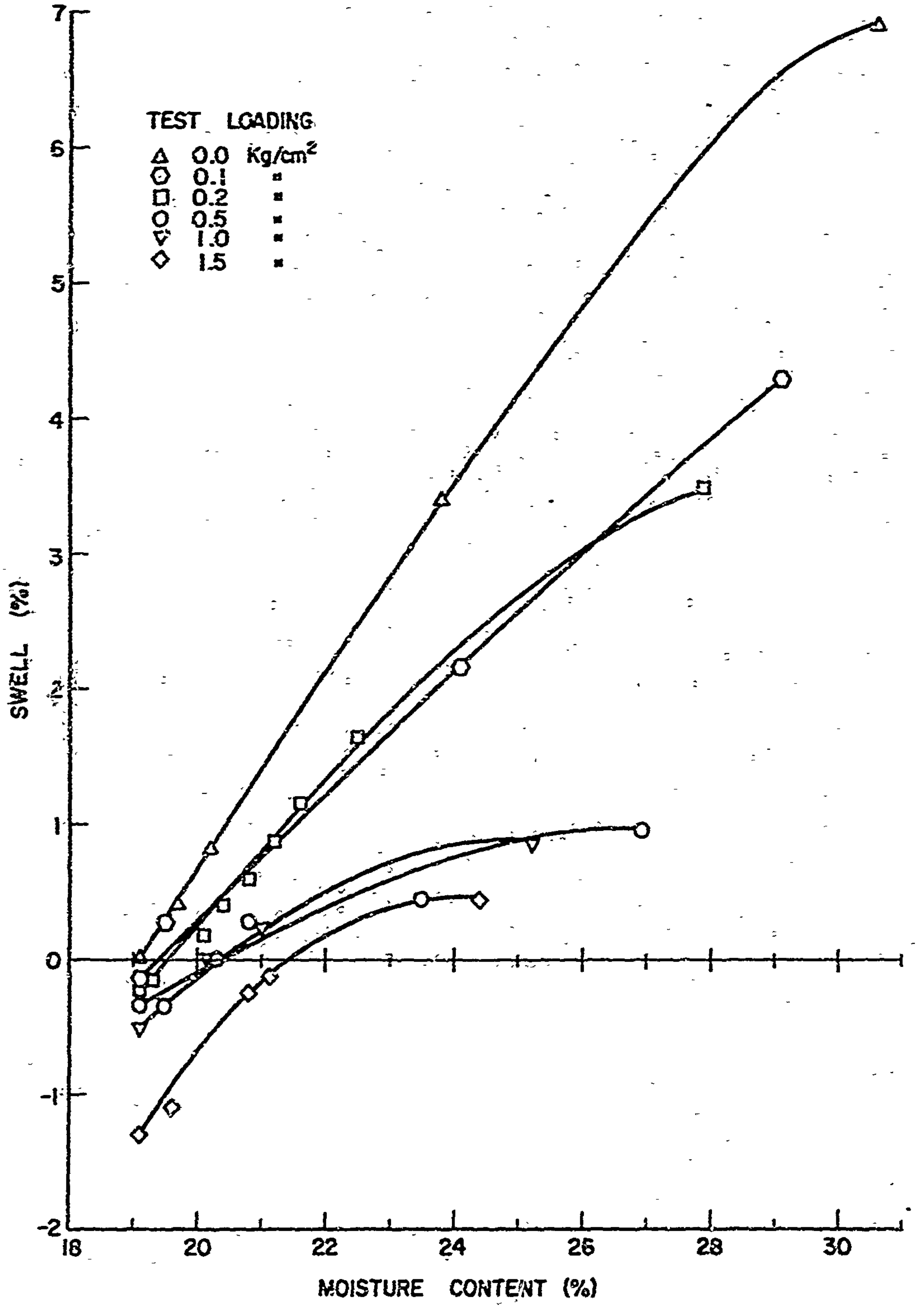


Figure 30. Effect of Increasing Moisture Content on Swell for Initial Moisture Content of 19.1%

examine what occurs in the pores as the suction pressures are reduced. As has been pointed out earlier, the soil compacted dry of optimum probably has an incomplete development of the clay micelle. The relative strengths of the various components (capillary, osmotic and adsorbed water) are not known exactly but as water is applied to the soil during the swell test, the capillary tensions in the soil pores are reduced. Then, during the equilibrium period a certain drop in the soil suction occurs. Since the capillary forces are acting to hold the soil particles together, they in a sense retard the swell. With a release of these pressures, the soil particles may imbibe water into the clay micelle which will increase the repulsive forces and cause the swell to start.

Another factor which would be active in this range of suction pressures is the elastic rebound of clay particles which have been deformed by the high suction pressures. As the suction pressure is reduced, these particles may regain their initial position and thereby cause some volume expansion.

The departure from the straight part of the curve also corresponds very closely with the point at which the swell begins to increase rapidly with change in moisture content (Fig. 29). There would appear to be some critical value of soil suction at which a higher rate of swell would occur under a given loading. This point corresponds to a condition where the forces pushing particles apart (osmotic) start to greatly exceed the combination of the load and suction forces forcing the particles together. As the size of the

load increased, the soil suction vs swell curve approached a straight line and did not swell as rapidly with a change in moisture content.

Below this critical value, the swell that occurs could be almost entirely due to the water required to equalize the ion concentration between the pore water and the water in the clay micelle.

The osmotic swell has been termed a "repulsive pressure" since as a result of imbibing of additional water molecules, the particles will be pushed further apart. Depending upon the size of the applied load, the degree of expansion indicates the equilibrium between the force (applied load) pushing particles together and the force (osmotic pressure) trying to push the particles apart. The osmotic demand was very strong since the imbibed water was deionized. There exists a large inbalance between the clay micelle and the pore water. As the water molecules are taken into the clay micelle, the micelle increases in size and swell must obviously occur.

The final degree of saturation was in all cases less than 100% at zero soil suction. This has been observed by Gibbs (23) and Matyas (49) to be the case for soils at higher initial soil suctions.

The occurrence of entrapped air is a possible cause of incomplete saturation. Due to the higher amount of air voids initially the chance of entrapment is much greater than in a soil compacted with a wetter initial condition.

<u>Swell test results for samples compacted at 26.0%</u> The data obtained from the swell test for the 26% moisture content sample are

presented graphically in Figs. 31, 32, and 33. Total swell for these samples was considerably less than for the dry side compacted samples. The maximum swell under zero loading was 0.9% and this was suppressed by even light loadings. When the load exceeded 0.5 kg/cm^2, consolidation occurred, and no swelling occurred as the moisture content increased slightly. A loading of 1.0 kg/cm^2 was performed but continued consolidation occurred and the sample reached approximately full saturation during the equilibrium period between the applied load and measured pore water pressures.

The soil suction and moisture content relationships under test loading are shown in Fig. 31. The final water content for a zero loading was 28.2% at a zero suction. The moisture content at zero suction decreased with increasing loading to 26.9% under 1.0 kg/cm^2. Table 4 summarizes the swell data and experimental results are contained in Appendix III.

<u>Discussion of results for swell tests (26%)</u> As expected, the swell for the samples compacted wet of optimum was considerably less than those on the dry side of optimum content for the same dry density.

The swell that occurs in samples compacted wet of optimum moisture content is primarily due to osmotic expansion. Mitchell (14) and others (15) have shown that swell in this moisture range could be completely prevented by using pore water of high concentrations of calcium acetate and calcium chloride.

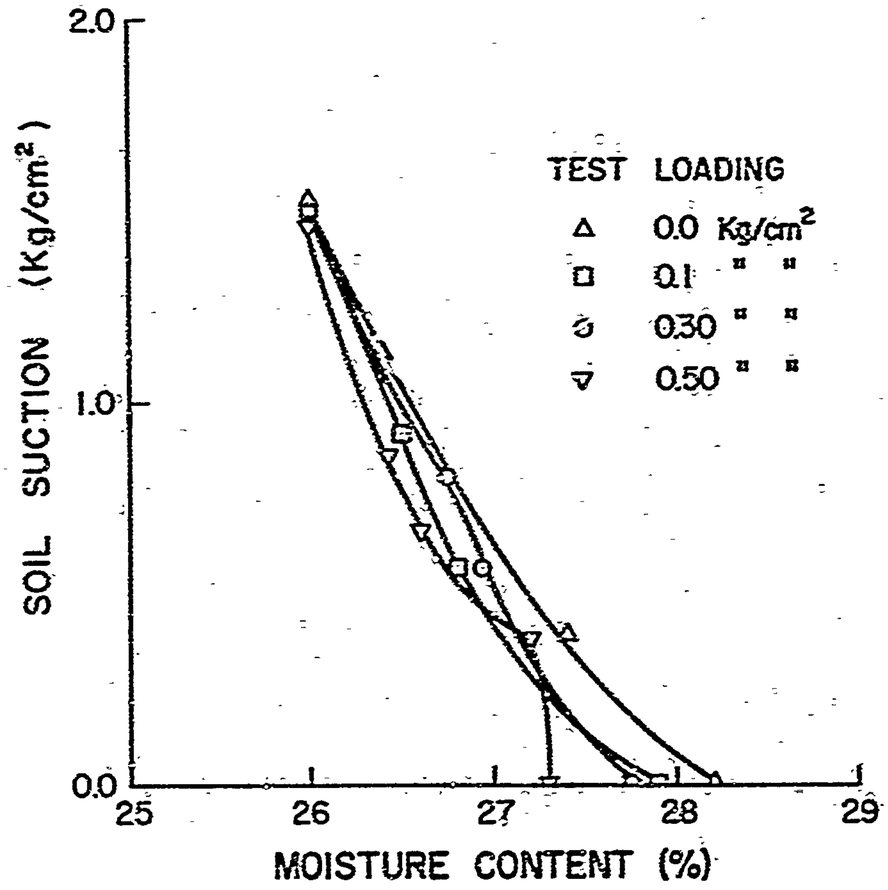


Figure 31. Effect of Increasing Moisture Content on Soil Suction for Initial Moisture Content of 26%

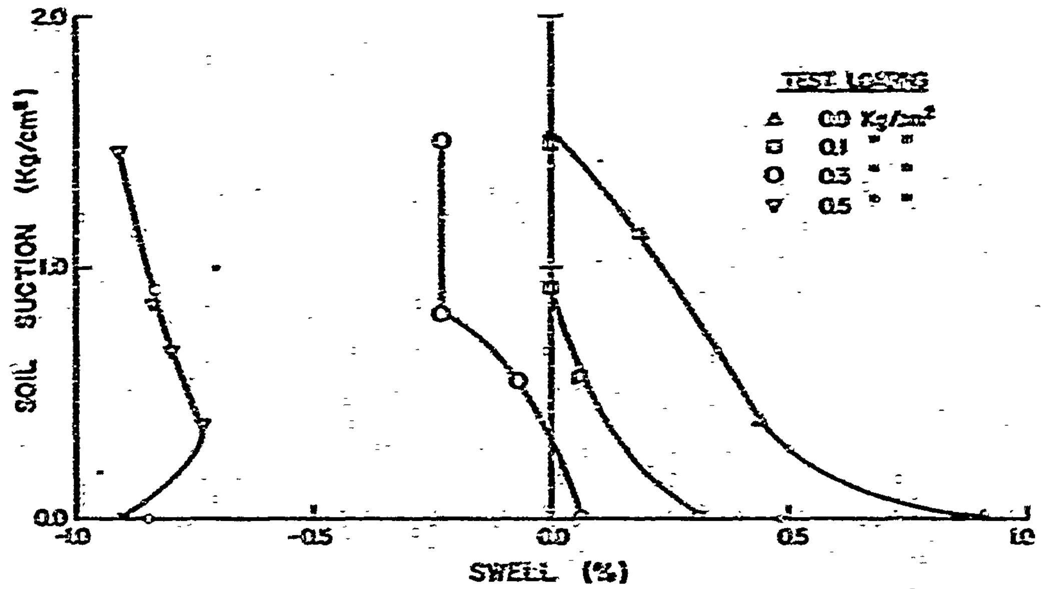


Figure 32. Effect of Decreasing Soil Suction on Swell for Initial Moisture Content of 26%

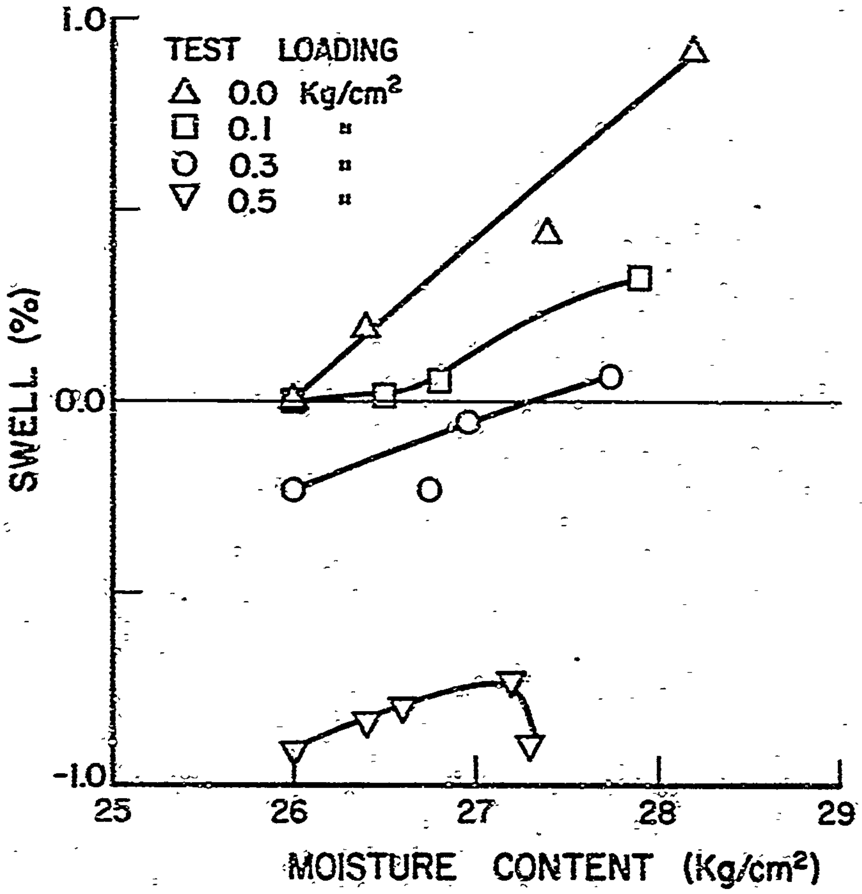


Figure 33. Effect of Increasing Moisture Content on Swell for Initial Moisture Content of 26%

TABLE 4 SWELL DATA SUMMARY 26.0% INITIAL MOISTURE CONTENT

Applied loading, kg/cm^2	0.0	0.1	0.3	0.5	1.0
Initial soil suction, kg/cm^2	1.53	1.50	1.50	1.46	1.00
Initial degree of saturation, (%)	98.3	98.4	98.9	99.9	100.0
Initial moisture content, (%)	26.0	26.0	26.0	26.0	26.0
Final moisture content, (%)	28.2	27.9	27.7	27.3	26.9
Final degree of saturation, (%)	100	100	100	100	100
Swell, (%)	0.91	0.31	0.05	-0.9	-1.4

The initial soil suctions were much less (1.5 kg/cm^2) as the adsorbed films were completely satisfied. The soil suction reflects the osmotic demand ~~almost~~ entirely.

The shape of the soil suction vs moisture content curves is much less uniform than for the 19.1% samples. However, there is still a similarity in the 0.0 and 0.10 kg/cm^2 load curves.

The sample loaded with 0.3 kg/cm^2 did not swell after equilibrium had been reached (initial consolidation) until the soil suction was reduced to about 0.80 kg/cm^2. Total swell was only 0.1% at a zero suction. The sample under a loading of 0.5 kg/cm^2 had an initial consolidation of about 0.9%. The sample then began to swell slightly as the soil suction was decreased. At some point below a suction of 0.38 kg/cm^2 the sample collapsed and consolidated to about the initial equilibrium volume. The sample loaded with 1.0 kg/cm^2 applied load continued to consolidate as the suction was reduced.

The final degree of saturation as calculated was slightly over 100% and can be attributed to the inability of determining the exact specific gravity of a soil. The specific gravity as measured is only an average value. In a clay soil, it is subject to errors due to the fact that oven drying to 110°C does not drive off all the adsorbed water. The value then determined is partly due to the mineral and partly due to the small film of water remaining around the clay particle.

SECTION VI

SUMMARY

Mineralogy and Engineering Index Properties The San Saba Clay contained 51.3% by weight of clay size particles of which a significant amount was montmorillonite with lesser amounts of kaolinite and illite. The presence of montmorillonite in large quantities is a good indication that the soil will be expansive. The engineering index properties showed the soil to have a liquid limit of 58 and a plasticity index of 36.4 which would also indicate the soil has at least a moderate swelling potential.

Microstructure The microstructure for specimens compacted on the wet and dry side of optimum moisture content (at equal dry densities) by kneading compaction does not appear to differ as much as previously thought. The degree of preferred particle orientation was very small and both microstructures were still within the range of flocculent microstructures. For the San Saba Clay the highest preferred orientation was obtained near optimum moisture content and remained constant until very high moisture contents were reached. Above a moisture content of 26% the degree of dispersion begins to decrease.

From the scanning electron micrographs obtained of samples at the two moisture contents, it appeared that the microstructure was characterized by a multi-grain or packet type of orientation. The

high molding water content sample had a higher degree of packet orientation and was much less open in appearance.

Microstructure and Swell The attempt to establish the influence of soil microstructure on swelling was not successful, primarily because the difference in microstructures was not significant for the soil tested. However, two methods for identifying the microstructure have been developed and should prove valuable in further studies of the soil microstructure.

For the two microstructures produced by compaction, the orientation ratio of 0.52 for the wet sample is still in the general range of a flocculent structure, and while having an increased degree of orientation it does not approach any high degree of dispersed orientation. Certainly the large differences in swell are almost entirely related to the initial water demand (soil suction) as opposed to small differences in microstructure.

Swell Test The swell test developed has several advantages over the current standard tests. The most important factor is being able to measure and control the soil suction. The technique used was successful both in being able to measure soil suctions up to 75 psi under loaded conditions as well as controlling the release of the suction.

The volumetric expansion due to entrapped air is minimized by this test method as water is applied only from one side. In the

standard swell test, the soil specimen is submerged and swelling caused by entrapped air could be a factor which greatly influences the results obtained. The values obtained by use of the Bishop Oedometer will minimize this effect and should give more realistic results, as well as a more complete idea of the amount of swell at various stages of saturation. This, of course, is important where the gradual increase of moisture occurs under a structure. Methods available to estimate the equilibrium moisture content expected in the field, or the seasonal fluctuation, could then be used with this test method to place the soil in the predicted final state and then measure the resulting swell. It would seem that this method could also be used to simulate seasonal variances and thereby be useful to run a short term "environmental" study of the soil in question. While the test period would perhaps cover several weeks to months, this would still be more practical than a number of years as is now required to evaluate the seasonal influences on a soil.

Swell and Soil Suction It is felt that this research has opened the door to a new method to evaluate swell in which the influence of the environment (seasonal wetting and drying) can for the first time be simulated in the laboratory. The influence of soil suction upon the swelling behavior of a soil is most important and is very dependent upon the initial value of the suction. As has been shown by comparison of two initial states (moisture contents) the resultant swell varies greatly. The soil suction acts as a stress, tending to give the soil rigidity, which also tends to repress the swell. As

the soil suction is reduced this force holding particles together becomes less and the repulsive forces become greater by the interactions of the clay micelles. There appears to be some critical value of soil suction where the repulsive or swelling forces overcome the forces holding particles together and swell will increase very rapidly with a further reduction of soil suction. This point is also influenced by the applied load on the soil. The lower the initial suction pressures (wetter the soil) the less swell will result due to the more fully developed clay micelles.

The importance of the initial state of the soil (or more precisely the initial soil suction) is very important. For a change in moisture content of say 2% the resulting swell will vary considerably depending upon the initial soil suction. The data for the drier sample (higher initial soil suction) indicate that the majority of the swell (approximately 70%) will occur after the soil suction is reduced below 3.2 kg/cm^2 for the light loadings (Fig. 34). When the load applied was 0.5 kg/cm^2 or larger the total swell was retarded and the soil suction vs swell curve approaches a straight line.

A similar trend is evident from the results of the wet sample. A soil suction of 0.4 kg/cm^2 apparently is the critical value from which swell increases at a higher rate. (Loads 0.0 and 0.1.) The amount of swell that occurs with soil suctions less than 0.4 kg/cm^2 is much less proportionately compared to the drier sample.

Load vs Change in Initial Soil Suction The effect on the initial soil suction of increasing the loading was shown in Fig. 35.

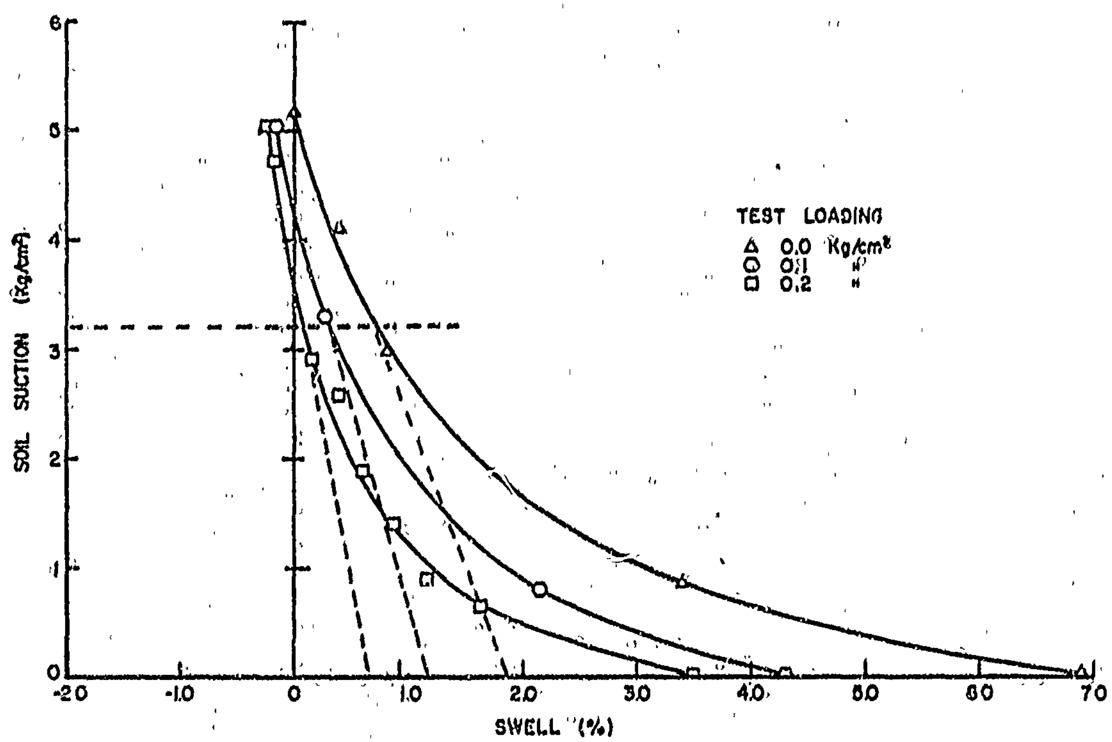


Figure 34. Effect of Soil Suction on the Amount of Swell

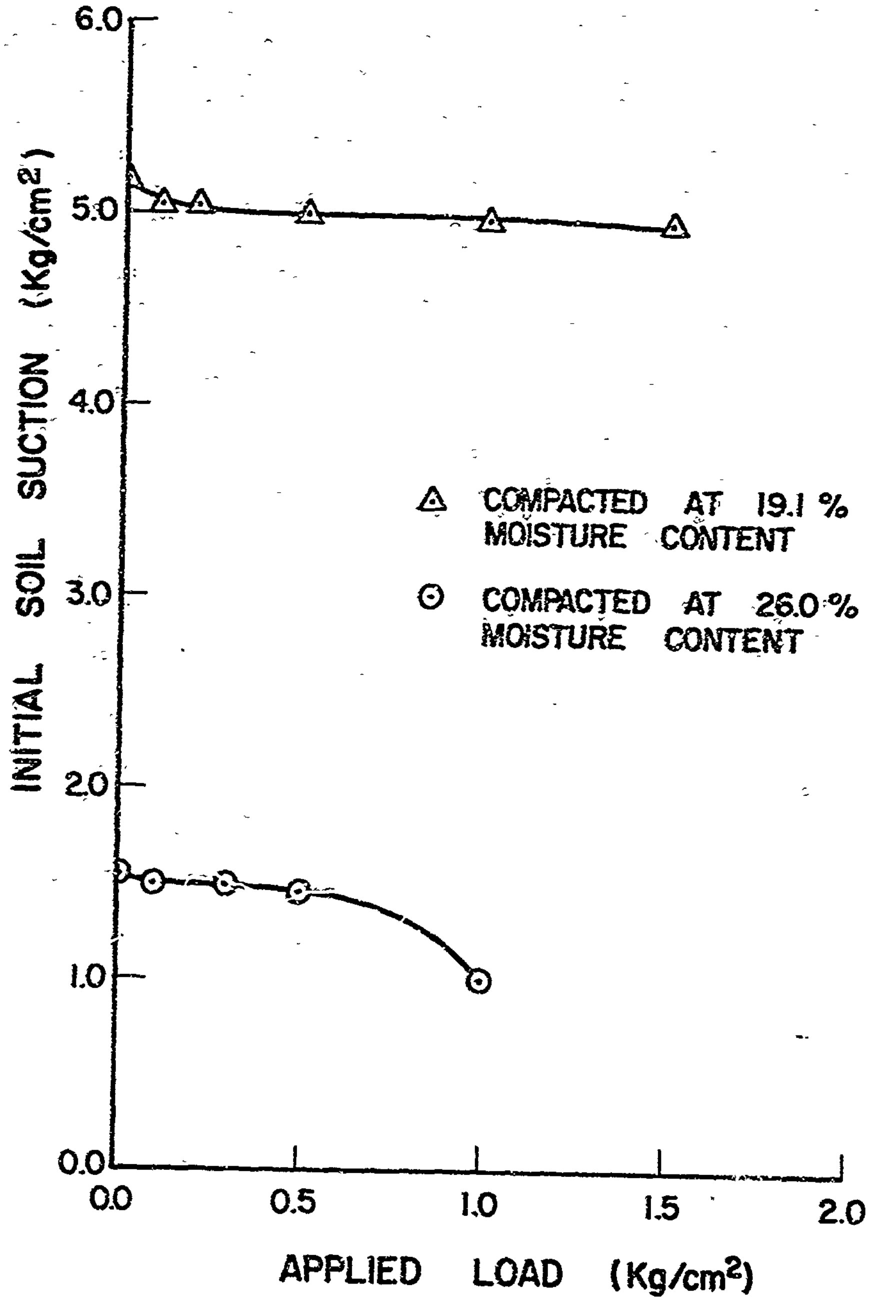


Figure 35. Effect of Applied Load on the Initial Soil Suction

In general, the effect was small. The change in the initial soil suction is probably closely related to the amount of consolidation the sample undergoes during load application. As consolidation occurs, the soil mass becomes denser and particles are closer together. This would have a tendency to reduce the soil suction slightly. The only test sample to undergo large consolidation was the one compacted at 26% when subjected to a 1.0 kg/cm^2 load. A drop in the initial soil suction of 0.5 kg/cm^2 occurred.

SECTION VII

CONCLUSIONS

1. Both the X-ray technique and scanning electron microscopy indicate that the soil microstructure does not change significantly with moisture content when kneading compaction is used. This is contrary to information presently available in the literature.
2. A packet type of microstructure is a more descriptive term to describe the soil microstructure. The packets in sample wet of optimum appear to have a greater degree of dispersion than samples at drier moisture content. From the X-ray technique the highest degree of dispersion was obtained near optimum moisture content. Very wet of optimum the degree of dispersion began to decrease slightly.
3. The swell test developed was successful in measuring and controlling soil suctions under applied external loadings to values of 5.1 kg/cm^2 (pF=3.4). The results for the soil tested indicate that there was some value of soil suction below which the degree of swelling accelerated.
4. The data obtained indicate that the applied loading has little effect upon the initial soil suction. Only when large consolidations occurred was there a significant change in the initial soil suction.
5. As the magnitude of applied loading was increased the amount of swell decreased. Increasing the applied loading made the soil suction vs swell relationship more linear and decreased the amount of swell as the soil suction approached zero.

SECTION VIII

RECOMMENDATIONS

1. The microstructure should be studied for additional soils to further verify that packet rather than single grain microstructures are prevalent. The use of the Scanning Electron Microscope would appear to be a valuable tool which could be used in this study.
2. Further research should be undertaken to better understand the swell-soil suction relationship. Undisturbed soil samples should be studied by reproducing the in situ conditions and then studying the swelling behavior as the soil suction is varied.
3. The study of environmental or cyclic effect of varying soil suctions under loaded conditions by the use of the swell test developed would be of value to the soils engineer.
4. The range of soil suctions studied should be extended using higher air entry ceramic stones. This swell test could be utilized for suctions up to a pF of 4.7, which is the design limit of the Bishop Oedometer.

APPENDIX I

X-RAY DIFFRACTION PATTERNS
FOR SAN SABA AND KAOLINITE CLAYS

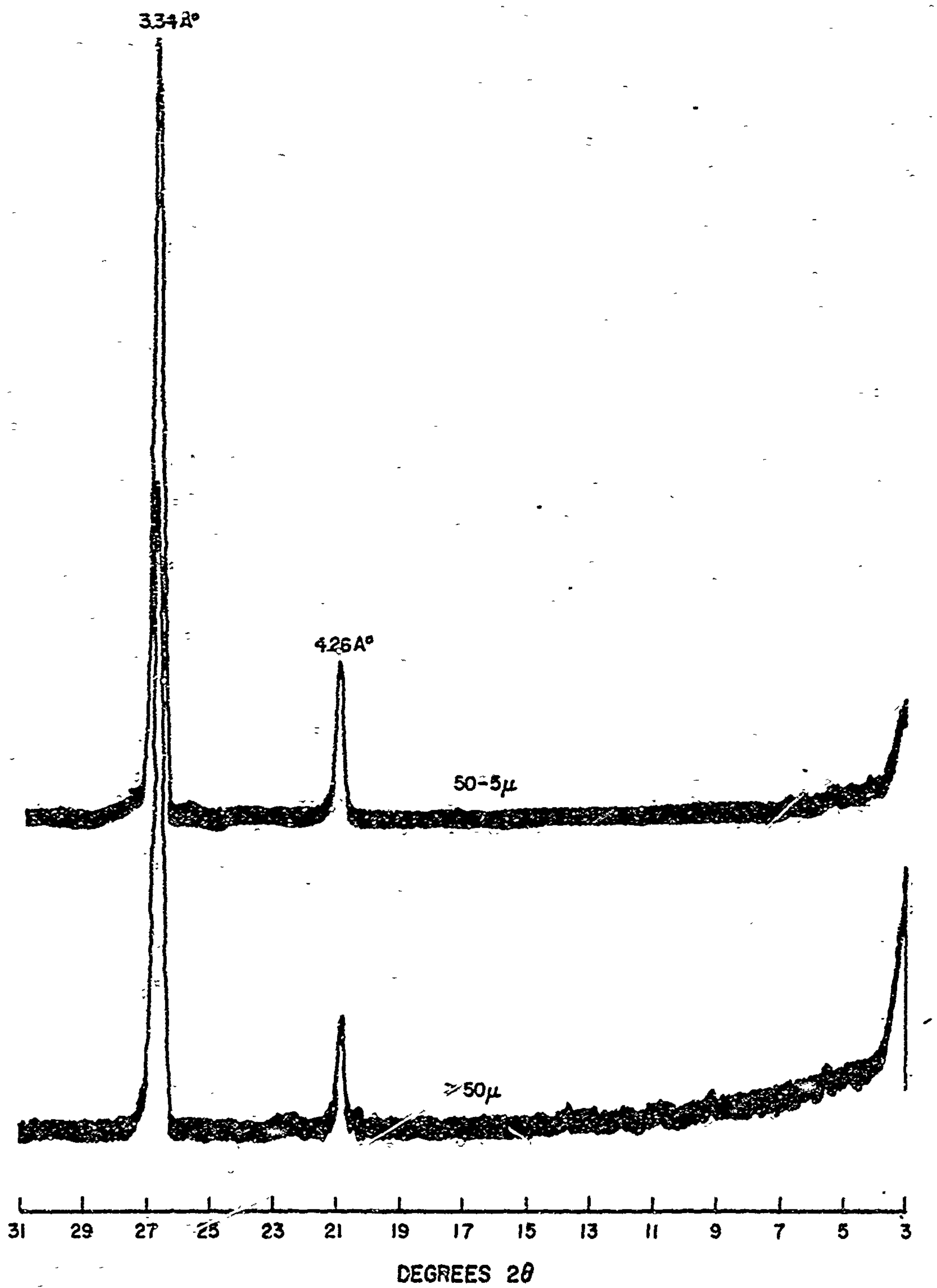


Figure 36. X-Ray Diffraction Patterns of San Saba Clay, Sand and Silt Fractions

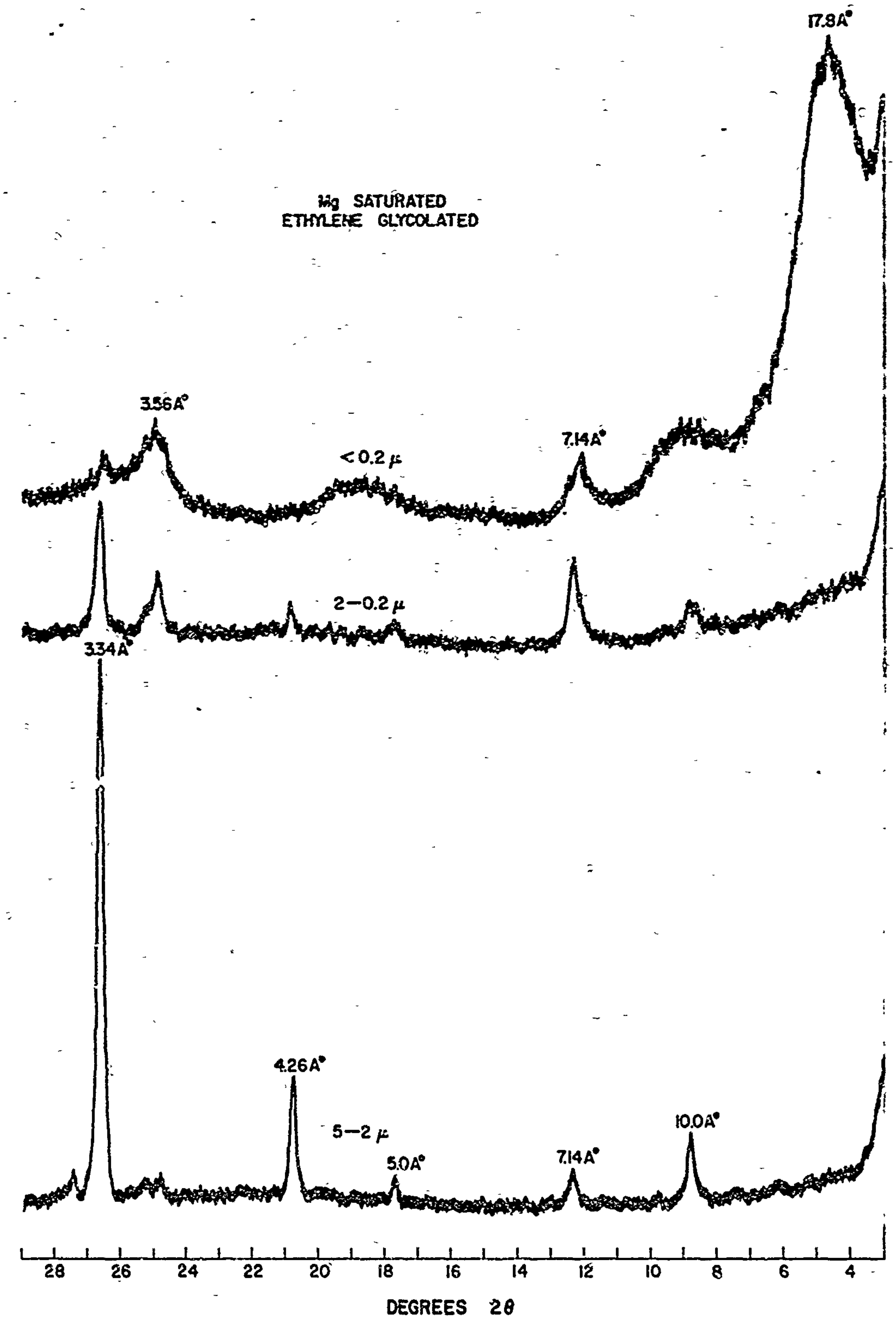


Figure 37. X-Ray Diffraction Patterns of San Saba Clay, Fine Silt and Clay Fractions

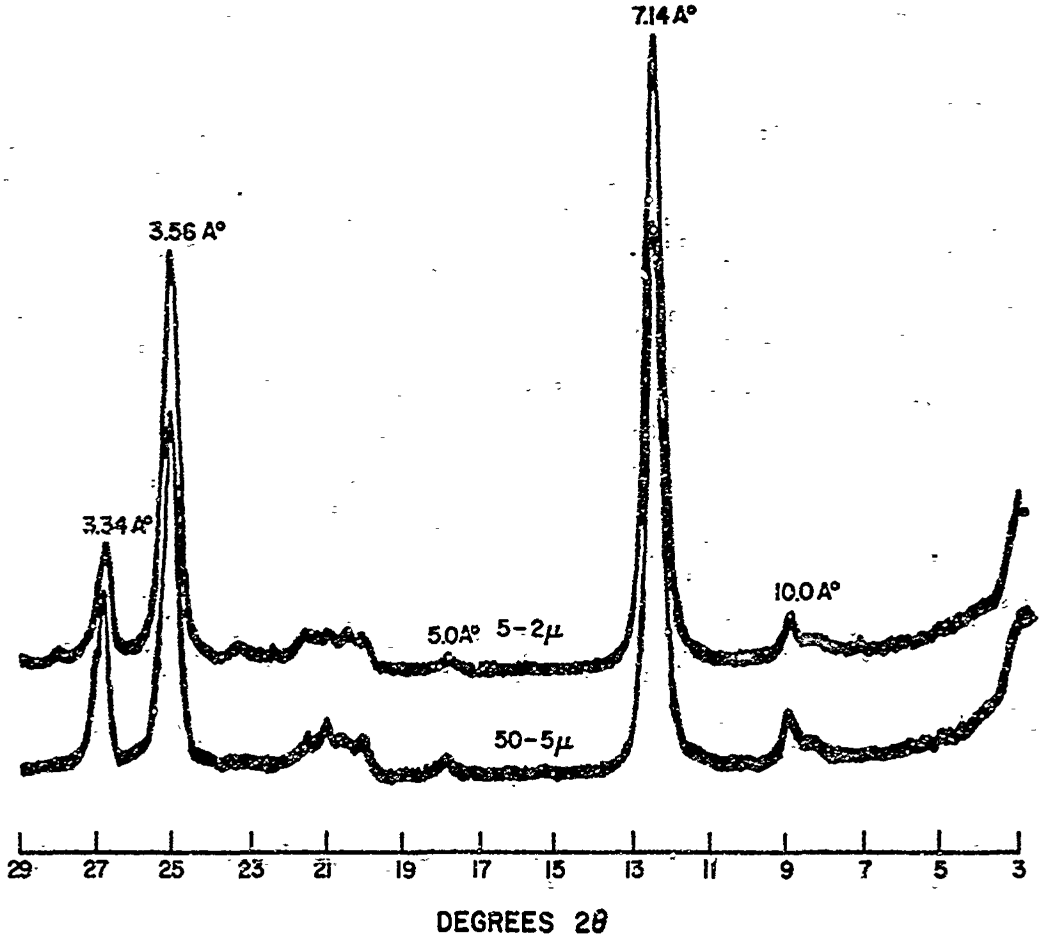


Figure 38. X-Ray Diffraction Patterns of Kaolinite, Sand and Silt Fractions

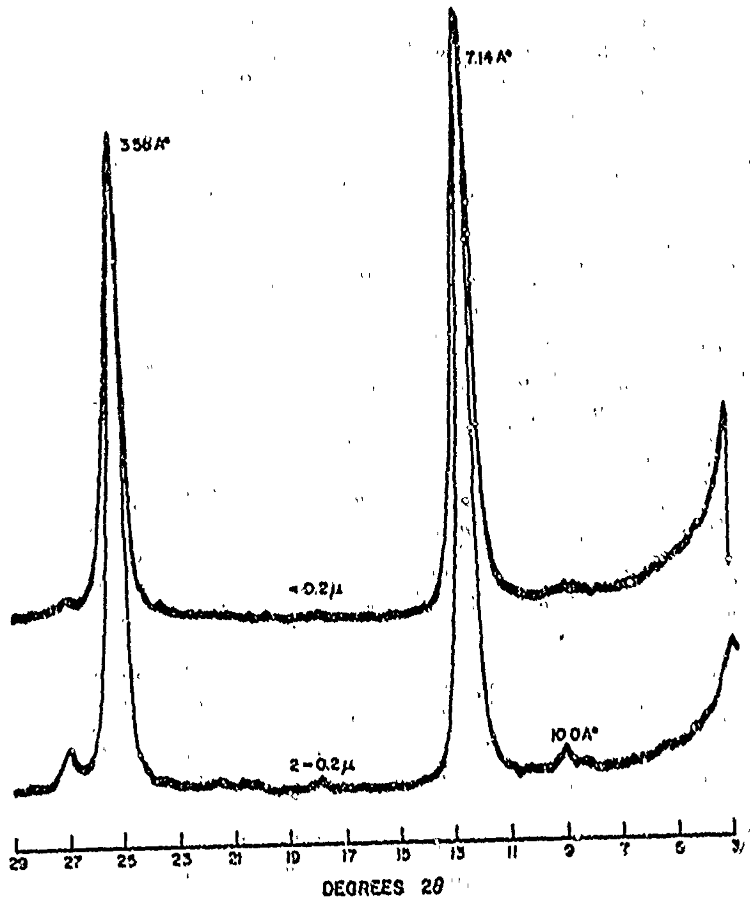


Figure 39. X-Ray Diffraction Patterns of Kaolinite, Clay Fractions

APPENDIX II

X-RAY DIFFRACTION DATA FOR MICROSTRUCTURE

TABLE 5 ORIENTATION RATIO DATA

Moisture content, %	Peak ratio (parallel)	Peak ratio (Perpendicular)	Orientation ratio
16.5	0.715	0.855	0.84
16.5	0.667	0.820	0.81
19.1	0.630	0.840	0.75
19.0	0.398	0.534	0.75
19.1	0.780	1.040	0.75
22.0	0.535	1.030	0.52
22.0	0.600	1.050	0.57
26.0	0.445	0.895	0.50
26.0	0.477	0.910	0.52
26.0	0.590	1.120	0.53
27.5	0.394	0.640	0.62
27.5	0.538	0.888	0.61

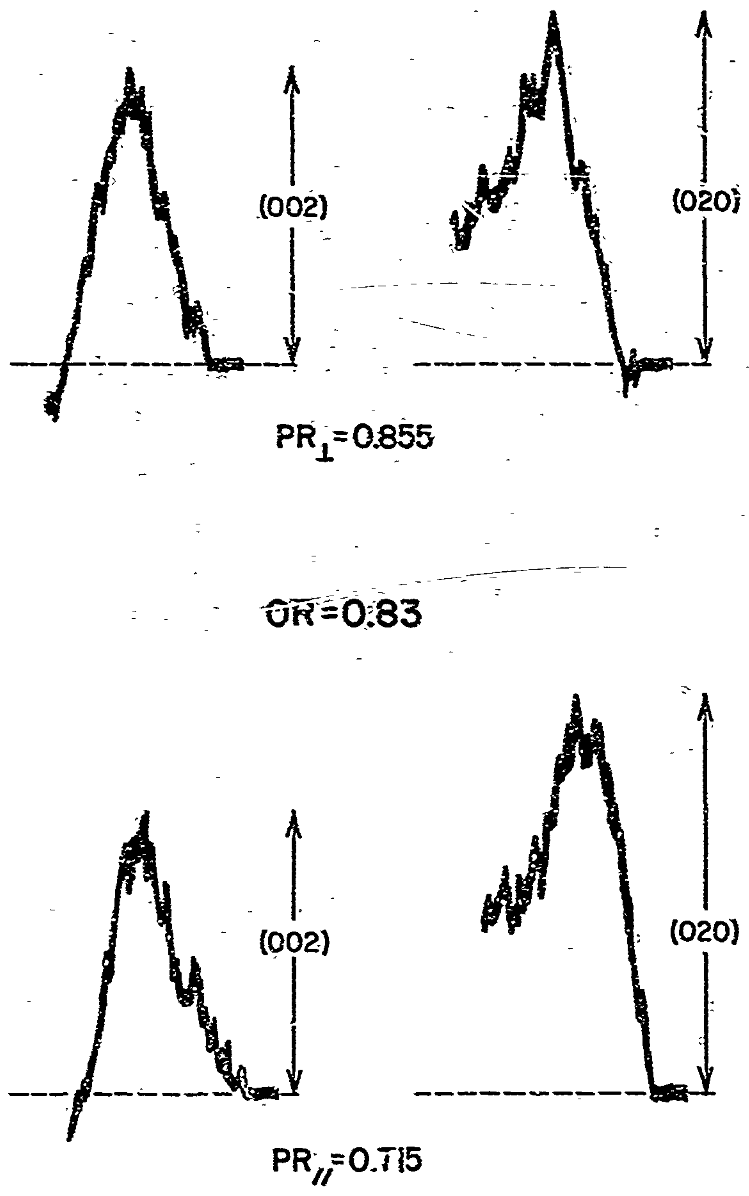


Figure 40. Orientation Ratio for Compacted Sample at a Moisture Content of 16.5%

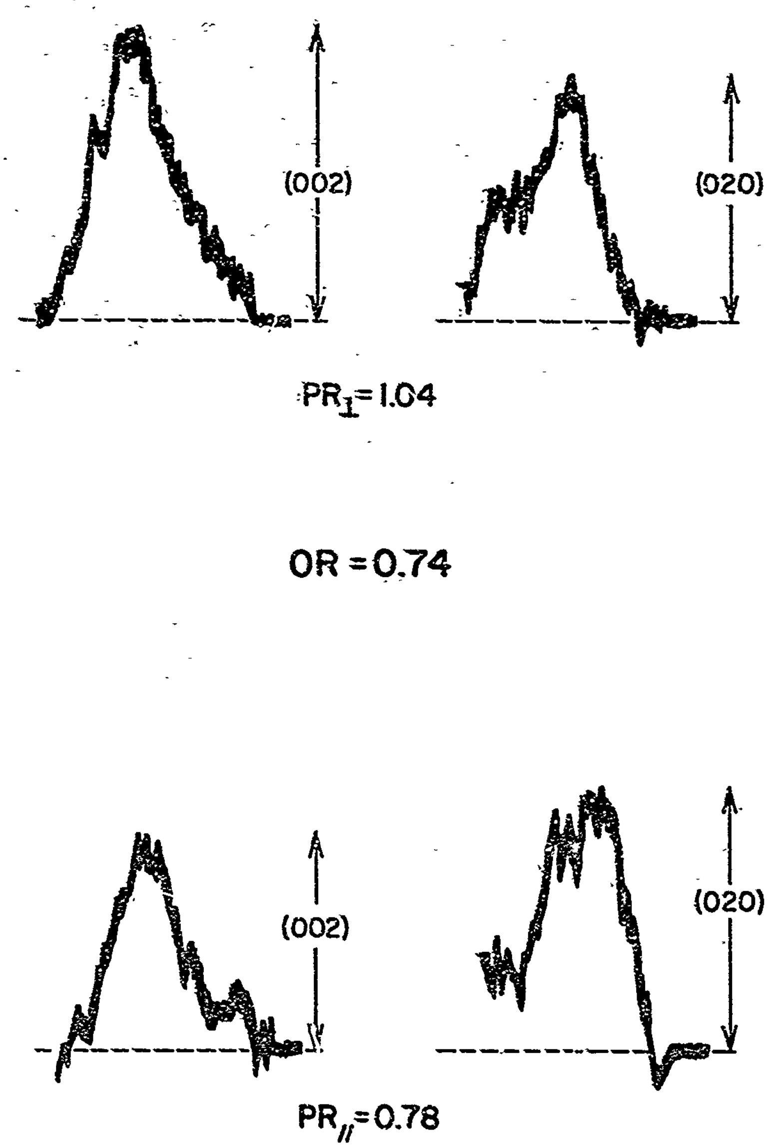


Figure 41. Orientation Ratio for Compacted Sample at a Moisture Content of 19.1%

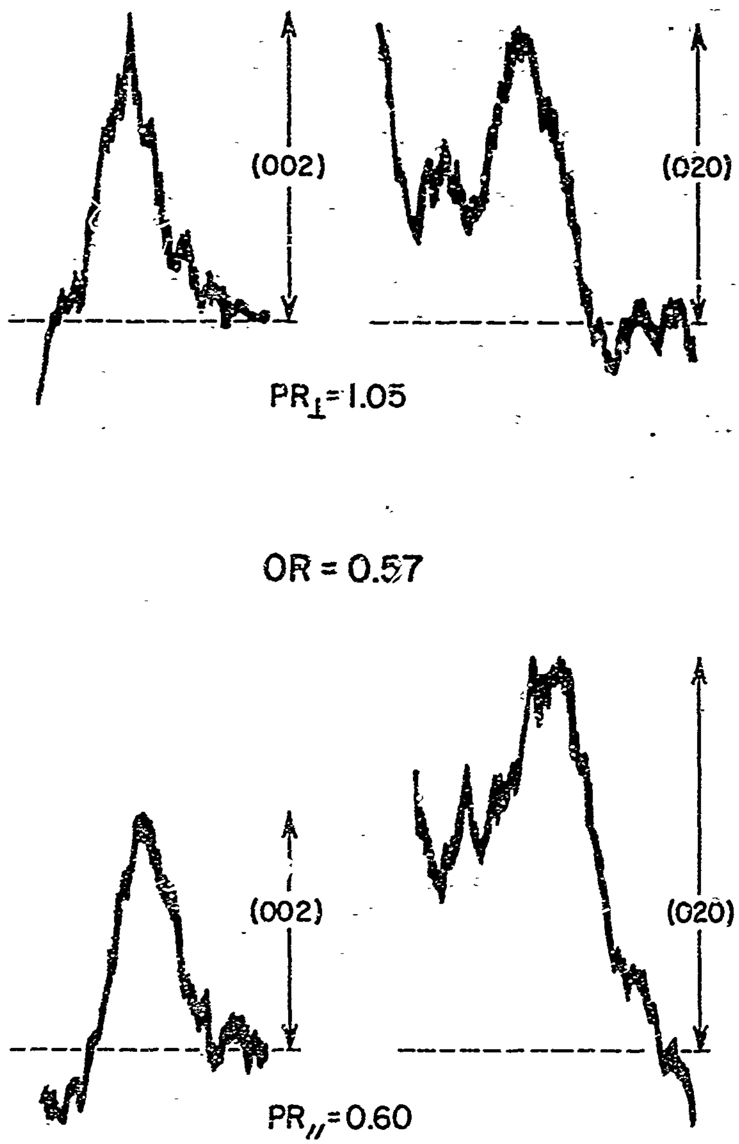


Figure 42. Orientation Ratio for Compacted Sample at a Moisture Content of 22.0%

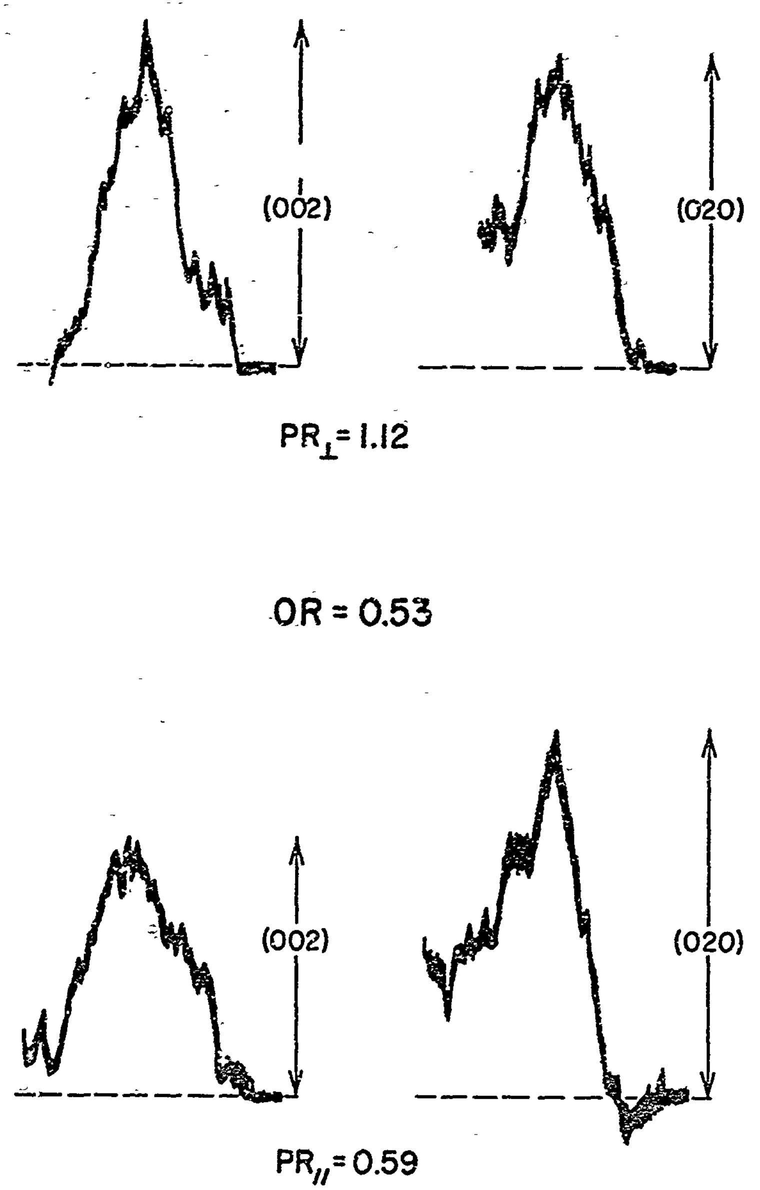


Figure 43. Orientation Ratio for Compacted Sample at a Moisture Content of 26%

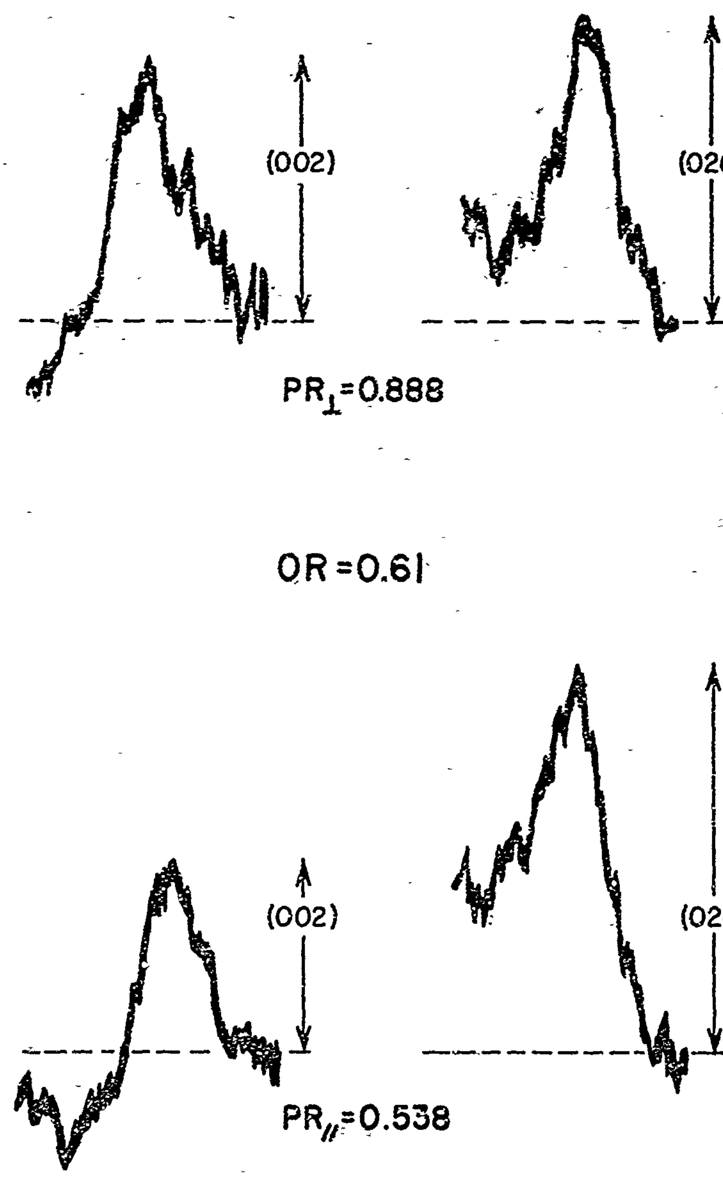


Figure 44. Orientation Ratio for Compacted Sample at a Moisture Content of 27.5%

APPENDIX III

SWELL TEST EXPERIMENTAL DATA

SHELL TEST DATA

Load 0.00 Dry Density 96.6 Water Content 19.1

U_a = Applied Air Pressure U_w = Measured Pore Water Pressure

U_c = Soil Suction = $U_a - U_w$

Elapsed Time	Soil Suction U_a	U_w	U_c	Back Press	Dial Rdg.	ΔH	Water Scale	Δ Water
(min.)	(Kg/cm²)				(0.0001 in)		(cc)	
0	4.8				2350.5	0		
285	4.8	0.16	4.64		2350.5	0		
880	4.8	-0.38	5.18		2351.0	0		
				Back Pressure Applied				
0				4.8	2351	0	3.0	0
5				4.8	2351	0	3.3	0.3
20				4.8	2350	0	3.8	0.8
				Back Pressure Removed				
0	4.8	0.0	4.8	0.0	2350	1		
170	4.8	3.44	1.36	0.0	2337	13		
420	4.8	0.85	3.95	0.0	2322	28		
630	4.8	0.80	4.00	0.0	2321	30		
				Back Pressure Applied				
0				4.8	2321	30	3.8	0.8
10				4.8	2320	31	4.0	1.0
30				4.8	2318	32	4.4	1.4
35				4.8	2318	33	4.5	1.5
				Back Pressure Removed				
0	4.8	4.8	0	0.0	2318	33		
775	4.8	1.8	3.0	0.0	2290	61		
1395	4.8	1.8	3.0	0.0	2290	61		
				Back Pressure Applied				
0				4.8	2290	61	4.5	1.5
5				4.8	2283	68	4.7	1.7
203				4.8	2167	84	9.2	6.2
				Back Pressure Removed				
0	4.8	4.8	0	0.0	2167	184		
720	4.8	3.95	0.85	0.0	2116	235		

Elapsed Time	Soil Suction			Back Press.	Dial Rdg.	ΔH	Water Scale	Δ Water
	U_a	U_w	U_c					
(min.)	(Kg/cm²)				(0.0001 in)		(cc)	
			Back Pressure Applied					
0				4.8	2116	235	9.2	6.2
80				4.8	2100	251	10.5	7.5
205				4.8	2061	290	12.4	9.4
390				4.8	1986	365	14.6	11.6
450				4.8	1960	391	14.9	11.9
520				4.8	1936	415	15.2	12.1
0				4.8	1932	419	4.3	12.2
55				4.8	1917	434	4.5	12.4
780				4.8	1868	483	5.9	13.8
1030				4.8	1851	500	6.2	14.1
1220				4.8	1848	503	6.4	14.3
1455				4.8	1839	512	6.5	14.4
1620				4.8	1832	519	6.6	14.5
1420				4.8	1831	520	7.1	15.0
1705				4.8	1831	520	7.1	15.0
			Back Pressure Removed					
0	4.8	4.8	0.0	0.0	1831	520		
100	4.8	4.8	0.0	0.0	1831	520		

SWELL TEST DATA

Load 0.1 Dry Density 96.8 Water Content 19.1

U_a = Applied Air Pressure U_w = Measured Pore Water Pressure

U_c = Soil Suction = $U_a - U_w$

Elapsed Time	Soil Suction U_a	U_w	U_c	Back Press	Dial Rdg.	ΔH	Water Scale	Δ Water
(min.)	(Kg/cm²)				(0.0001 in)		(cc)	
0	4.8			0	0037	0		
1	4.8			0	0042	-5		
5	4.8			0	0046	-9		
20	4.8	0.70	4.10	0	0048	-11		
180	4.8	0.15	4.65	0	0048	-11		
270	4.8	0.24	5.04	0	0048	-11		
410	4.8	0.24	5.04	0	0048	-11		
			Back Pressure	Applied				
0				4.8	0048	-11	3.0	0
90				4.8	0041	-4	4.5	1.5
			Back Pressure	Removed				
0	4.8	4.8	0	0	0041	-4		
610	4.8	1.85	2.95	0	0033	4		
850	4.8	1.50	3.30	0	0017	20		
1010	4.8	1.50	3.30	0	0016	21		
			Back Pressure	Applied				
0				4.8	0016	21	4.5	1.5
280				4.8	2487	50	9.8	6.7
			Back Pressure	Removed				
0	4.8	4.8	0	0	2487	50		
790	4.8	4.0	0.8	0	2378	159		
1120	4.8	4.0	0.8	0	2377	160		
1380	4.8	4.0	0.8	0	2375	162		

Elapsed Time	Soil Suction			Back Press	Dial Rdg.	ΔH	Water Scale	Δ Water
	U_a	U_w	U_c					
(min.)	(Kg/cm²)				(0.0001 in)		(cc)	
			Back Pressure Applied					
0				4.8	2375	162	9.8	6.7
775				4.8	2272	265	15.2	12.1
							re zero scale	
775				4.8	2272	265	4.6	12.1
1050				4.8	2259	278	5.0	12.5
1220				4.8	2249	288	5.1	12.6
2470				4.8	2217	320	6.0	13.5
3790				4.8	2220	337	6.6	14.1
4090				4.8	2184	333	7.2	14.7

SHELL TEST DATA

Load 0.2 Dry Density 96.8 Water Content 19.1

U_a = Applied Air Pressure U_w = Measured Pore Water Pressure

U_c = Soil Suction = $U_a - U_w$

Elapsed Time	Soil Suction			Back Press	Dial Rdg.	ΔH	Water Scale	Δ Water
	U_a	U_w	U_c					
(min.)	(Kg/cm²)				(0.0001 in)		(cc)	
0	4.8			0	0644	0		
3	4.8			0	0653	-9		
5	4.8	3.70	1.10	0	0654	-10		
10	4.8	3.65	1.15	0	0654	-10		
25	4.8	3.20	1.60	0	0655	-11		
40	4.8	2.85	1.95	0	0655	-11		
160	4.8	1.2	3.60	0	0659	-15		
370	4.8	0.13	4.67	0	0660	-16		
490	4.8	0.20	5.00	0	0660	-16		
700	4.8	0.20	5.00	0	0660	-16		
			Back Pressure Applied					
0				4.8	0660	-16	1.9	0.0
10				4.8	0660	-16	2.2	0.3
			Back Pressure Removed					
0	4.8	4.8	0.0	0	0660	-16		
510	4.8	0.3	4.5	0	0658	-14		
920	4.8	0.06	4.74	0	0657	-13		
1160	4.8	0.06	4.74	0	0657	-13		
			Back Pressure Applied					
0				4.8	0657	-13	2.2	0.3
3				4.8	0654	-10	2.4	0.5
35				4.8	0649	-5	2.7	0.8
			Back Pressure Removed					
0	4.8	4.8	0	0	0649	-5		
140	4.8	2.9	1.9	0	0646	-2		
740	4.8	1.2	3.6	0	0645	-1		
1050	4.8	1.06	3.74	0	0644	0		
1200	4.8	1.06	3.74	0	0644	0		
1385	4.8	1.06	3.74	0	0644	0		

Elapsed Time	Soil Suction U_a	U_w	U_c	Back Press	Dial Rdg.	ΔH	Water Scale	Δ Water
(min.)	(Kg/cm²)				(0.0001 in)		(cc)	
			Back Pressure Applied					
0				4.8	0644	0	2.7	0.8
15				4.8	0639	5	3.0	1.1
35				4.8	0637	7	3.1	1.2
45				4.8	0636	8	3.2	1.3
			Back Pressure Removed					
0	4.8	4.8	0	0	0636	8		
735	4.8	2.1	2.7	0	0630	14		
1000	4.8	1.95	2.85	0	0630	14		
1080	4.8	1.90	2.90	0	0630	14		
1220	4.8	1.90	2.90	0	0630	14		
1775	4.8	1.90	2.90	0	0630	14		
			Back Pressure Applied					
0				4.8	0630	14	3.2	1.3
45				4.8	0623	21	3.6	1.7
			Back Pressure Removed					
0	4.8	4.8	0	0	0623	21		
910	4.8	2.5	2.3	0	0617	27		
2270	4.8	2.22	2.58	0	0615	29		
			Back Pressure Applied					
0				4.8	0615	29	4.0	1.7
40				4.8	0610	34	4.3	2.0
65				4.8	0608	36	4.45	2.2
70				4.8	0608	36	4.5	2.3
			Back Pressure Removed					
0	4.8	4.8	0	0	0608	36		
200	4.8	3.72	1.08	0	0604	40		
520	4.8	3.40	1.40	0	0602	42		
610	4.8	3.3	1.50	0	0601	43		
1590	4.8	2.92	1.88	0	0599	45		
			Back Pressure Applied					
0				4.8	0599	45	4.7	2.3
20				4.8	0596	48	4.9	2.5
90				4.8	0591	53	5.3	2.9

Elapsed Time	Soil Suction U_a	U_w	U_c	Back Press	Dial Rdg.	ΔH	Water Scale	Δ Water
(min.)	(Kg/cm^2)				(0.0001 in)		(cc)	
			Back Pressure Removed					
0	4.8	4.8	0	0	0591	53		
90	4.8	4.18	0.62	0	0588	56		
1320	4.8	3.45	1.35	0	0579	65		
1540	4.8	3.40	1.40	0	0578	66		
1740	4.8	3.40	1.40	0	0578	66		
			Back Pressure Applied					
0				4.8	0578	66	5.3	2.9
15				4.8	0576	68	5.4	3.0
80				4.8	0574	70	5.6	3.2
130				4.8	0571	73	5.75	3.3
155				4.8	0569	75	5.8	3.4
			Back Pressure Removed					
0	4.8	4.8	0	0	0569	75		
800	4.8	3.95	0.85	0	0559	85		
920	4.8	3.90	0.90	0	0558	86		
1075	4.8	3.90	0.90	0	0557	87		
			Back Pressure Applied					
0				4.8	0557	87	6.2	3.4
50				4.8	0554	90	6.5	3.7
80				4.8	0552	92	6.7	3.9
140				4.8	0541	103	7.4	4.6
			Back Pressure Removed					
0	4.8	4.8	0	0	0541	103		
225	4.8	4.3	0.5	0	0533	111		
835	4.8	4.16	0.64	0	0524	120		
1075	4.8	4.15	0.65	0	0523	121		

Elapsed Time	Soil Suction U_a	U_w	U_c	Back Press	Dial Rdg.	ΔH	Water Scale	Δ Water
(min.)	(Kg/cm²)				(0.0001 in)		(cc)	
			Back Pressure Applied					
0				4.8	0523	121	3.3	4.6
1410				4.8	0440	204	8.5	9.8
1520				4.8	0416	228	9.2	10.5
1790				4.8	0412	232	9.3	10.6
1970				4.8	0410	234	9.4	10.7
2030				4.8	0409	235	9.4	10.7
2180				4.8	0407	237	9.55	10.8
2975				4.8	0400	244	9.8	11.1
3260				4.8	0398	246	9.9	11.2
3440				4.8	0397	247	9.9	11.2
3770				4.8	0394	250	10.0	11.3
5850				4.8	0388	256	10.3	11.6
6930				4.8	0383	261	10.5	11.8
7230				4.8	0382	262	10.5	11.8

SWELL TEST DATA

Load 0.5 Dry Density 96.8 Water Content 19.1

U_a = Applied Air Pressure U_w = Measured Pore Water Pressure

U_c = Soil Suction = $U_a - U_w$

Elapsed Time	Soil Suction			Back Press	Dial Rdg.	ΔH	Water Scale	Δ Water
	U_a	U_w	U_c					
(min.)	(Kg/cm²)				(0.0001 in)		(cc)	
0	4.8			0	0778	0		
1	4.8			0	0789	-11		
5	4.8			0	0795	-17		
20	4.8	2.9	1.9	0	0802	-24		
60	4.8	2.2	2.7	0	0802	-24		
180	4.8	1.5	3.3	0	0802	-24		
360	4.8	0.17	4.63	0	0802	-24		
480	4.8	0.11	4.91	0	0802	-24		
600	4.8	0.19	4.99	0	0802	-24		
720	4.8	0.20	5.00	0	0802	-24		
			Back Pressure Applied					
0				4.8	0802	-24	1.1	0.0
5				4.8	0802	-24	1.3	0.2
10				4.8	0800	-22	1.6	0.5
			Back Pressure Removed					
0	4.8	4.8	0	0	0800	-22		
510	4.8	0.51	4.29	0	0800	-22		
840	4.8	0.32	4.48	0	0800	-22		
			Back Pressure Applied					
0				4.8	0800		1.6	0.5
25				4.8	0797		2.7	1.6
			Back Pressure Removed					
0	4.8	4.8	0	0	0797	-19		
90	4.8	3.7	1.1	0	0791	-13		
240	4.8	3.25	1.55	0	0784	-6		
350	4.8	2.75	2.05	0	0781	-3		
880	4.8	1.95	2.85	0	0778	0		
			Back Pressure Applied					
0				4.8	0778	0	2.7	1.6
5				4.8	0778	0	3.0	1.9
15				4.8	0775	3	3.4	2.3

Elapsed Time	Soil Suction U_a	U_w	U_c	Back Press	Dial Rdg.	ΔH	Water Scale	Δ Water
(min.)	(Kg/cm^2)				(0.0001 in)		(cc)	
			Back Pressure Removed					
0	4.8	4.8	0	0	0775	3		
315	4.8	3.7	1.1	0	0770	8		
1440	4.8	3.0	1.80	0	0756	22		
			Back Pressure Applied					
0				4.8	0756	22	3.4	2.3
30				4.8	0755	23	6.1	5.0
55				4.8	0751	27	7.0	5.9
			Back Pressure Removed					
0	4.8	4.8	0	0	0751	27		
1440	4.8	4.3	0.5	0	0744	34		
			Back Pressure Applied					
0				4.8	0744	34	7.0	5.9
660				4.8	0731	47	8.2	7.1
1410				4.8	0724	54	10.5	9.4
1890				4.8	0721	57	11.0	9.9
2880				4.8	0713	65	11.3	10.2
3360				4.8	0710	68	11.4	10.3
4680				4.8	0706	72	11.6	10.5

SWELL TEST DATA

Load 1.0 Dry Density 96.6 Water Content 19.1

U_a = Applied Air Pressure U_w = Measured Pore Water Pressure

U_c = Soil Suction = $U_a - U_w$

Elapsed Time	Soil Suction U_a	Soil Suction U_w	Soil Suction U_c	Back Press	Dial Rdg.	ΔH	Water Scale	Δ Water
(min.)	(Kg/cm²)				(0.0001 in)		(cc)	
0	4.8			0	0270			
1	4.8			0	0280	-10		
5	4.8			0	0286	-16		
10	4.8			0	0290	-20		
20	4.8	2.8	2.0	0	0295	-25		
60	4.8	2.2	2.6	0	0300	-30		
95	4.8	1.7	3.1	0	0303	-33		
140	4.8	1.45	3.35	0	0304	-34		
245	4.8	0.90	3.90	0	0306	-36		
325	4.8	0.21	4.59	0	0306	-36		
480	4.8	0.01	4.79	0	0307	-37		
600	4.8	0.14	4.94	0	0308	-38		
715	4.8	0.19	4.99	0	0308	-38		
835	4.8	0.19	4.99	0	0308	-38		
			Back Pressure Applied					
0				4.8	0308	-38	3.1	0
5				4.8	0305	-35	3.4	0.3
10				4.8	0302	-32	3.5	0.4
15				4.8	0300	-30	3.6	0.5
20				4.8	0300	-30	3.7	0.6
			Back Pressure Removed					
0	4.8	4.8	0	0	0300	-30		
315	4.8	1.85	2.95	0	0295	-25		
435	4.8	1.60	3.20	0	0295	-25		
615	4.8	1.45	3.35	0	0295	-25		
695	4.8	1.15	3.65	0	0295	-25		
805	4.8	1.00	3.80	0	0295	-25		
895	4.8	0.90	3.90	0	0295	-25		
1055	4.8	0.90	3.90	0	0295	-25		
1975	4.8	0.90	3.90	0	0295	-25		

Elapsed Time	Soil Suction U_a	U_w	U_c	Back Press	Dial Rdg.	ΔH	Water Scale	Δ Water
(min.)	(Kg/cm²)				(0.0001 in)		(cc)	
			Back Pressure	Applied				
0				4.8	0295	-25	3.7	0.6
5				4.8	0291	-21	3.9	0.8
55				4.8	0282	-12	4.6	1.5
			Back Pressure	Removed				
0	4.8	4.8	0	0	0282	-12		
610	4.8	2.7	2.10	0	0272	-2		
750	4.8	2.5	2.30	0	0270	0		
900	4.8	2.4	2.40	0	0269	1		
1035	4.8	2.25	2.55	0	0268	2		
1260	4.8	2.10	2.70	0	0268	2		
1390	4.8	2.10	2.70	0	0268	2		
			Back Pressure	Applied				
0				4.8	0268	2	4.6	1.5
10				4.8	0268	2	4.9	1.8
150				4.8	0260	10	5.6	2.5
			Back Pressure	Removed				
0	4.8	4.8	0	0	0260	10		
595	4.8	3.70	1.10	0	0254	16		
845	4.8	3.50	1.30	0	0254	16		
1010	4.8	3.45	1.35	0	0253	17		
1425	4.8	3.40	1.40	0	0253	17		
			Back Pressure	Applied				
0				4.8	0253	17	5.6	2.5
600				4.8	0231	39	8.9	5.8
960				4.8	0223	47	10.1	7.0
2220				4.8	0212	58	10.8	7.7
3600				4.8	0206	64	11.2	8.1

SHELL TEST DATA

Load 1.5 Dry Density 96.9 Water Content 19.1

U_a = Applied Air Pressure U_w = Measured Pore Water Pressure

U_c = Soil Suction = $U_a - U_w$

Elapsed Time	Soil Suction U_a	U_w	U_c	Back Press	Dial Rdg.	ΔH	Water Scale	Δ Water
(min.)	(Kg/cm²)				(0.0001 in)		(cc)	
0	4.8			0	0572	0		
1/4	4.8			0	0660	-88		
½	4.8			0	0664	-92		
1	4.8			0	0666	-94		
2	4.8			0	0667	-95		
4	4.8			0	0668	-96		
5	4.8			0	0669	-97		
15	4.8	2.4	2.4	0	0670	-98		
30	4.8	1.2	3.6	0	0670	-98		
60	4.8	0.58	4.22	0	0670	-98		
835	4.8	0.16	4.96	0	0670	-98		
			Back Pressure Applied					
0				4.8	0670	-98	2.1	0.0
5				4.8	0667	-95	2.3	0.2
10				4.8	0664	-94	2.4	0.3
25				4.8	0657	-85	2.6	0.5
30				4.8	0655	-83	2.7	0.6
35				4.8	0653	-81	2.8	0.7
			Back Pressure Removed					
0	4.8	4.8	0	0	0653	-81		
30	4.8	3.7	1.1	0	0649	-77		
90	4.8	3.02	1.78	0	0647	-75		
140	4.8	2.45	2.35	0	0646	-74		
250	4.8	1.65	3.15	0	0646	-74		
370	4.8	1.25	3.55	0	0646	-74		
1210	4.8	0.54	4.26	0	0645	-73		
			Back Pressure Applied					
0				4.8	0645	-73	2.9	0.7
20				4.8	0638	-66	3.3	1.1
45				4.8	0633	-61	3.5	1.3
150				4.8	0614	-42	4.6	2.4

Elapsed Time	Soil Suction			Back Press	Dial Rdg.	ΔH	Water Scale	Δ Water
	U_a	U_w	U_c					
(min.)	(Kg/cm²)				(0.0001 in)		(cc)	
			Back Pressure	Removed				
0	4.8	4.8	0	0	0614	-42		
190	4.8	4.0	0.8	0	0602	-30		
360	4.8	3.9	0.9	0	0598	-26		
440	4.8	3.9	0.9	0	0596	-24		
535	4.8	3.75	1.05	0	0595	-23		
1185	4.8	3.4	1.4	0	0591	-19		
1350	4.8	3.35	1.45	0	0591	-19		
1590	4.8	3.35	1.45	0	0591	-19		
1860	4.8	3.35	1.45	0	0590	-18		
			Back Pressure	Applied				
0				4.8	0590	-18	4.8	2.4
30				4.8	0587	-15	5.2	2.8
			Back Pressure	Removed				
0	4.8	4.8	0	0	0587	-15		
705	4.8	4.0	0.8	0	0583	-11		
1020	4.8	3.70	1.1	0	0582	-10		
1215	4.8	3.70	1.1	0	0582	-10		
1500	4.8	3.65	1.1	0	0582	-10		
			Back Pressure	Applied				
0				4.8	0582	-10	5.3	2.8
60				4.8	0579	-7	5.8	3.3
660				4.8	0560	12	8.4	5.9
960				4.8	0557	15	8.7	6.2
1380				4.8	0554	18	8.9	6.4
2340				4.8	0549	23	9.4	7.0
2940				4.8	0547	25	9.5	7.0
3150				4.8	0547	25	9.6	7.1

SWELL TEST DATA

Load 0.0 Dry Density 96.8 Water Content 26%

U_a = Applied Air Pressure U_w = Measured Pore Water Pressure

U_c = Soil Suction = $U_a - U_w$

Elapsed Time	Soil Suction U_a	Soil Suction U_w	Soil Suction U_c	Back Press	Dial Rdg.	ΔH	Water Scale	Δ Water
(min.)	(Kg/cm²)				(0.0001 in)		(cc)	
0	1.5			0.0	0740			
630	1.5	-0.03	1.53	0.0	0740			
			Back Pressure	Applied				
0				1.5	0740	0	4.1	0
15				1.5	0739	1	4.3	0.2
60				1.5	0739	1	4.5	0.4
120				1.5	0737	3	4.6	0.5
160				1.5	0737	3	4.7	0.6
			Back Pressure	Removed				
0	1.5	1.5	0.0	0	0737	3		
125	1.5	1.41	0.09	0	0736	4		
230	1.5	0.88	0.62	0	0736	4		
350	1.5	0.45	1.05	0	0728	12		
500	1.5	0.37	1.13	0	0727	13		
590	1.5	0.37	1.13	0	0726	14		
			Back Pressure	Applied				
0				1.5	0726	14	4.7	0.6
180				1.5	0724	16	5.2	1.1
435				1.5	0722	18	5.9	1.8
			Back Pressure	Removed				
0	1.5	1.5	0.0	0	0722	18		
375	1.5	1.22	0.28	0	0713	27		
495	1.5	1.12	0.38	0	0707	33		
615	1.5	1.11	0.39	0	0707	33		
			Back Pressure	Applied				
0				1.5	0707	33	5.9	1.8
1500				1.5	0681	59	6.8	2.7
2880				1.5	0672	68	7.1	3.0

SWELL TEST DATA

Load 0.1 Dry Density 96.8 Water Content 26%

U_a = Applied Air Pressure U_w = Measured Pore Water Pressure

U_c = Soil Suction = $U_a - U_w$

Elapsed Time	Soil Suction U_a	U_w	U_c	Back Press	Dial Rdg.	ΔH	Water Scale	Δ Water
(min.)	(Kg/cm²)				(0.0001 in)		(cc)	
0	1.5			0	0550			
600	1.5	0.05	1.45	0	0550			
820	1.5	0.06	1.46	0	·0550			
			Back Pressure Applied					
0				1.5	0549	1.0	5.5	0.0
1				1.5	0549	1.0	5.7	0.2
20				1.5	0549	1.0	5.9	0.4
175				1.5	0549	0.5	6.2	0.7
			Back Pressure Removed					
0	1.5	1.5	0	0	0548	1.0		
125	1.5	1.35	0.15	0	0548	2.0		
260	1.5	0.90	0.60	0	0548	2.0		
445	1.5	0.775	0.72	0	0548	2.0		
755	1.5	0.58	0.92	0	0548	2.0		
			Back Pressure Applied					
0				1.5	0547	3.0	6.3	0.8
60				1.5	0547	3.0	6.5	1.0
180				1.5	0547	3.0	6.6	1.1
190				1.5	0547	3.0	6.6	1.1
			Back Pressure Removed					
0	1.5	1.5	0	0	0547	3.0		
215	1.5	1.13	0.37	0	0547	3.0		
420	1.5	0.96	0.54	0	0547	3.0		
575	1.5	0.94	0.56	0	0547	3.0		
			Back Pressure Applied				Re zero scale	
0				1.5	0547	3.0	2.9	1.1
195				1.5	0547	3.0	3.3	1.5
885				1.5	0536	14.0	3.7	1.9
1215				1.5	0536	14.0	3.9	2.1
1860				1.5	0531	19.0	4.2	2.4
2445				1.5	0528	22.0	4.4	2.6

SWELL TEST DATA

Load 0.3 Dry Density 96.8 Water Content 26%

U_a = Applied Air Pressure U_w = Measured Pore Water Pressure

U_c = Soil Suction = $U_a - U_w$

Elapsed Time	Soil Suction U_a	U_w	U_c	Back Press	Dial Rdg.	ΔH	Water Scale	Δ Water
(min.)	(Kg/cm²)				(0.0001 in)		(cc)	
0	1.5			0	1663			
4	1.5			0	1677	-14		
15	1.5			0	1679	-16		
30	1.5	0.36	1.14	0	1679	-16		
45	1.5	0.30	1.20	0	1679	-16		
60	1.5	0.27	1.23	0	1679	-16		
120	1.5	0.17	1.33	0	1679	-16		
250	1.5	0.06	1.44	0	1679	-16		
430	1.5	0.00	1.50	0	1679	-16		
525	1.5	0.00	1.50	0	1679	-16		
			Back Pressure	Applied				
0				1.5	1679	-16	4.6	0
0				1.5	1679	-16	4.7	0.1
0				1.5	1679	-16	4.8	0.2
1				1.5	1679	-16	5.0	0.4
2				1.5	1679	-16	5.1	0.5
4				1.5	1679	-16	5.3	0.7
8				1.5	1679	-16	5.5	0.9
14				1.5	1679	-16	5.6	1.0
			Back Pressure	Removed				
0	1.5	1.37	0	0	1679	-16		
16	1.5	1.37	0.13	0	1679	-16		
50	1.5	1.28	0.22	0	1679	-16		
150	1.5	1.13	0.37	0	1679	-16		
810	1.5	0.69	0.81	0	1679	-16		
			Back Pressure	Applied				
0				1.5	1679	-16	5.6	1.0
½				1.5	1679	-16	5.7	1.1
1				1.5	1679	-16	5.7	1.1
2				1.5	1679	-16	5.85	1.25
4				1.5	1678	-15	5.9	1.30

Elapsed Time	Soil Suction			Back Press	Dial Rdg.	ΔH	Water Scale	Δ Water
	U_a	U_w	U_c					
(min.)	(Kg/cm²)				(0.0001 in)		(cc)	
			Back Pressure Removed					
0	1.5	1.5	0	0	1678	-15		
10	1.5	1.31	0.19	0	1678	-15		
30	1.5	1.27	0.23	0	1678	-15		
65	1.5	1.24	0.26	0	1677	-14		
110	1.5	1.21	0.29	0	1676	-13		
355	1.5	1.10	0.40	0	1674	-11		
430	1.5	1.09	0.41	0	1674	-11		
650	1.5	1.02	0.48	0	1673	-10		
1310	1.5	0.93	0.57	0	1673	-10		
			Back Pressure Applied				Re zero scale	
0				1.5	1673	-10	6.2	1.3
5				1.5	1673	-10	6.5	1.6
40				1.5	1672	-9	6.6	1.7
100				1.5	1670	-7	6.7	1.8
355				1.5	1667	-4	6.8	1.9
680				1.5	1664	-1	6.9	2.0
740				1.5	1664	-1	6.9	2.0
1355				1.5	1660	3	7.2	2.2
1475				1.5	1660	3	7.2	2.2
1695				1.5	1660	3	7.3	2.3
1815				1.5	1660	3	7.3	2.3
2140				1.5	1659	4	7.4	2.4
2820				1.5	1658	5	7.4	2.4

SWELL TEST DATA

Load 0.5 Dry Density 96.8 Water Content 26%

U_a = Applied Air Pressure U_w = Measured Pore Water Pressure

U_c = Soil Suction = $U_a - U_w$

Elapsed Time	Soil Suction			Back Press	Dial Rdg.	ΔH	Water Scale	Δ Water
	U_a	U_w	U_c					
(min.)	(Kg/cm²)				(0.0001 in)		(cc)	
0	1.5			0	0680	0		
0	1.5			0	0720	-40		
5	1.5			0	0728	-48		
10	1.5			0	0735	-55		
15	1.5			0	0735	-55		
20	1.5			0	0737	-57		
30	1.5			0	0740	-60		
35	1.5	0.45	1.05	0	0740	-60		
75	1.5	0.34	1.16	0	0738	-58		
640	1.5	0.04	1.46	0	0750	-70		
			Back Pressure Applied					
0				1.5	0750		2.90	0
15				1.5	0748	-68	3.40	0.5
25				1.5	0747	-67	3.50	0.6
30				1.5	0747		3.50	0.6
			Back Pressure Removed					
0	1.5	1.5	0	0	0747	-67		
65	1.5	1.32	0.13	0	0745	-65		
370	1.5	0.89	0.61	0	0744	-64		
470	1.5	0.80	0.70	0	0743	-63		
680	1.5	0.69	0.81	0	0743	-63		
850	1.5	0.64	0.86	0	0743	-63		
			Back Pressure Applied					
0				1.5	0743	-63	3.5	0.6
20				1.5	0742	-62	3.9	1.0

Elapsed Time	Soil Suction			Back Press	Dial Rdg.	ΔH	Water Scale	Δ Water
	U_a	U_w	U_c					
(min.)	(Kg/cm^2)				(0.0001 in)		(cc)	
			Back Pressure Removed					
0	1.5	1.5	0.0	0	0742	-62		
530	1.5	0.94	0.56	0	0740	-60		
750	1.5	0.90	0.60	0	0740	-60		
825	1.5	0.88	0.62	0	0740	-60		
950	1.5	0.84	0.66	0	0740	-60		
1040	1.5	0.83	0.67	0	0740	-60		
			Back Pressure Applied					
0				1.5	0740	-60	3.9	1.0
50				1.5	0738	-58	4.2	1.3
130				1.5	0738	-58	4.5	1.5
			Back Pressure Removed					
0	1.5	1.5	0	0	0737	-57		
275	1.5	1.34	0.16	0	0737	-57		
825	1.5	1.28	0.24	0	0736	-54		
1060	1.5	1.18	0.32	0	0735	-55		
1410	1.5	1.12	0.38	0	0735	-55		
			Back Pressure Applied				Re zero	scale
0				1.5	0736	-54	4.7	1.6
300				1.5	0732	-52	5.0	1.9
1085				1.5	0729	-49	5.3	2.2
1140				1.5	0729	-49	5.3	2.2
1200				1.5	0728	-49	5.3	2.2
1560				1.5	0728	-49	5.4	2.3
2060				1.5	0728	-48	5.4	2.3

SWELL TEST DATA

Load 1.0 Dry Density 96.8 Water Content 26.0

U_a = Applied Air Pressure U_w = Measured Pore Water Pressure

U_c = Soil Suction = $U_a - U_w$

Elapsed Time	Soil Suction			Back Press	Dial Rdg.	ΔH	Water Scale	Δ Water
	U_a	U_w	U_c					
(min.)	(Kg/cm²)				(0.0001 in)		(cc)	
0	1.5			0	1717			
2	1.5			0	1800	-83		
5	1.5			0	1802	-85		
10	1.5			0	1805	-88		
20	1.5			0	1808	-91		
30	1.5	0.5	1.0	0	1810	-93		
80	1.5	0.52	0.98	0	1815	-98		
760	1.5	0.5	1.0	0	1825	-108		
			Back Pressure Applied					
0				1.5	1825	-108	6.1	0
5				1.5	1825	-108	6.3	0.2
15				1.5	1826	-109	6.3	0.2
45				1.5	1825	-108	6.3	0.2
130				1.5	1824	-107	6.4	0.3
			Back Pressure Removed					
130	1.5	1.5	0	0	1824	-107		
255	1.5	1.04	0.46	0	1824	-107		
325	1.5	0.97	0.53	0	1824	-107		
480	1.5	0.90	0.60	0	1824.3	-107		
660	1.5	0.80	0.70	0	1826	-109		
720	1.5	0.78	0.72	0	1826	-109		
790	1.5	0.76	0.74	0	1826	-109		
1450	1.5	0.66	0.84	0	1827	-110		
			Back Pressure Applied					
0				1.5	1827	-110	6.5	0.3
55				1.5	1826	-109	6.6	0.4
105				1.5	1826	-109	6.7	0.5
165				1.5	1826	-109	6.8	0.6
305				1.5	1824	-107	6.85	0.65
365				1.5	1823.5	-107	6.90	0.7
425				1.5	1823.5	-107	6.90	0.8
1420				1.5	1821.0	-104	7.25	1.05

Elapsed Time	Soil Suction			Back Press	Dial Rdg.	ΔH	Water Scale	Δ Water
	U_a	U_w	U_c					
(min.)	(Kg/cm²)				(0.0001 in)		(cc)	
1495				1.5	1821	-104	7.3	1.20
1645				1.5	1820	-103	7.3	1.10
1885				1.5	1820	-103	7.3	1.10
2365				1.5	1820	-103	7.4	1.20
2935				1.5	1820	-103	7.4	1.20

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13. ABSTRACT (Distribution Limitation Statement No. 2)

Influence of soil suction, clay microstructure, and applied external loading upon the swelling behavior was undertaken to further the understanding of expansive soils. For the soil tested, the microstructure produced by kneading compaction did not change the particle orientation to any significant degree. Evaluation of the microstructure was performed with both X-ray diffraction and scanning electron microscopy techniques. The microstructure is perhaps best described by multi-grain or packet arrangements. A higher degree of packet dispersion occurred at molding water contents wet of optimum; however, it did not approach any high degree of preferred particle orientation. A swell test was developed using the Bishop Oedometer in which both soil suction and external loading could be controlled and measured. Using the axis of translation technique with a high air entry ceramic stone soil suctions of 5.1 kg/cm² (pF 3.4) were measured. Results indicate that below a certain value of soil suction the swell will be accelerated with a further decrease in suction. As the applied loading is increased it will limit this acceleration and the soil suction-swell relationship becomes more linear with decreasing suctions.

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